McCONOUGHEY

Investigation of an Ore Crane

Civil Engineering

B. S.

1907

INVESTIGATION OF AN ORE CRANE

BY

EARL WYETH McCONOUGHEY

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

PRESENTED JUNE, 1907

C O L L E G E   O F   E N G I N E E R I N G

April 30, 1907.

This is to certify that the following thesis prepared under the immediate direction of Professor F. O. Dufour, Assistant Professor of Structural Engineering, by

EARL WYETH McCONOUGHEY

entitled INVESTIGATION OF AN ORE -Handling CRANE

is accepted by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O. Baker.

Head of Department of Civil Engineering

Introduction.

The greatest industry of the age is undoubtedly that of the production of iron and steel. With the almost incredible development, that this industry has undergone in the last twenty years, it is to be expected that the accompanying processes would develope on a more or less similar scale. Such has been the case with the handling of iron-ore. Up to a few years ago, all the ore used was unloaded by hand. An ore-boat when going up to the ore-dock, expected to remain there for a few weeks, whereas now, the time required for unloading is counted in hours. On all the principal docks, has been installed some form of mechanical unloader.

It is the purpose of this thesis to describe the ore-handling plant at South Chicago and investigate the members of the traveling gantry crane. The plant is located on the North Slip, at the South Works of the Illinois Steel Company.

Outline.

Art. 19. Compression - Box Girder.
20. Floor Beams.
21. Stringers.

Note. The pictures used as figures in this article were taken just after the completion of the second ore-bridge and consequently do not show the four ore-bridges which are now installed in this plant. They were published in an article that appeared in the "Iron Age" in the issue of Sept. 4, 1902; some of the data for this article was obtained from the same source.

Part I.

Description of the Ore-Handling Plant.

Art. 1. General Description.

The ore-handling plant was designed by Hoover and Mason, Contracting Engineers for Excavating and Conveying Machinery, Railway Exchange Building Chicago. The details were worked out and the entire plant fabricated at the North Works of the Illinois Steel Company.

The Iron Age for Sept. 4, 1902, says - "This ore-handling system is the most comprehensive thus far installed, including as it does every operation in handling ore at a furnace plant, from the unloading of the vessels by machinery to the filling of the stacks. ......

"The ingenuity displayed in accomplishing results, heretofore deemed impracticable to be undertaken by mechanical appliances, has elicited the unstinted commendation of the most experienced engineers.

"The practical utility - the saving of labor and money - of the new system

and ore-handling devices has been fully demonstrated to the satisfaction of the Illinois Steel Company and to the gratification of the designing engineers. When first adapted to the needs of one furnace, the invention was regarded as somewhat of an experiment in the way of ore-handling, but the fact that the system has been extended to another furnace, each having a daily capacity of 500 tons, is proof that it is a success.

"Under the old method of handling ore at South Chicago, from the boats at the dock to the charge at the furnace, over 300 men were employed for two furnaces, but the completion of the Hoover and Mason system permits the same amount of work to be done by 26, with little noise, no confusion and at a minimum cost."

Briefly, the plant consists of fifteen unloaders on the edge of the slip, four traveling gantry-cranes, and the ore pockets and storage yard. Each unloader has a five-ton bucket of the clam shell type, that runs out over the slip on a

pivoted arm. The grabs of the unloaders dump into cars on tracks directly below the unloaders, or into a large V-shaped trough just back of the tracks. The cars are used when it is wished to take the ore to furnaces other than those served by this plant. The trough is used for the ore intended for the six 500 ton furnaces, served directly by this plant. See Fig. 1. and 3.

The ore is taken from the trough by the ten-ton grabs of the four gantry-cranes and deposited in the storage yard or directly in the ore pockets as may be required. The ore-pockets are served by ~~an~~ electric laurys which run beneath them, and almost automatically take the required amounts of ore from the ore-pockets on one side and of coke and limestone from the pockets on the other side, for the furnace charges. The lauries deliver to skip hoists that automatically dump the charge into the charging bell at the top of the furnace stacks. See Fig. 2.

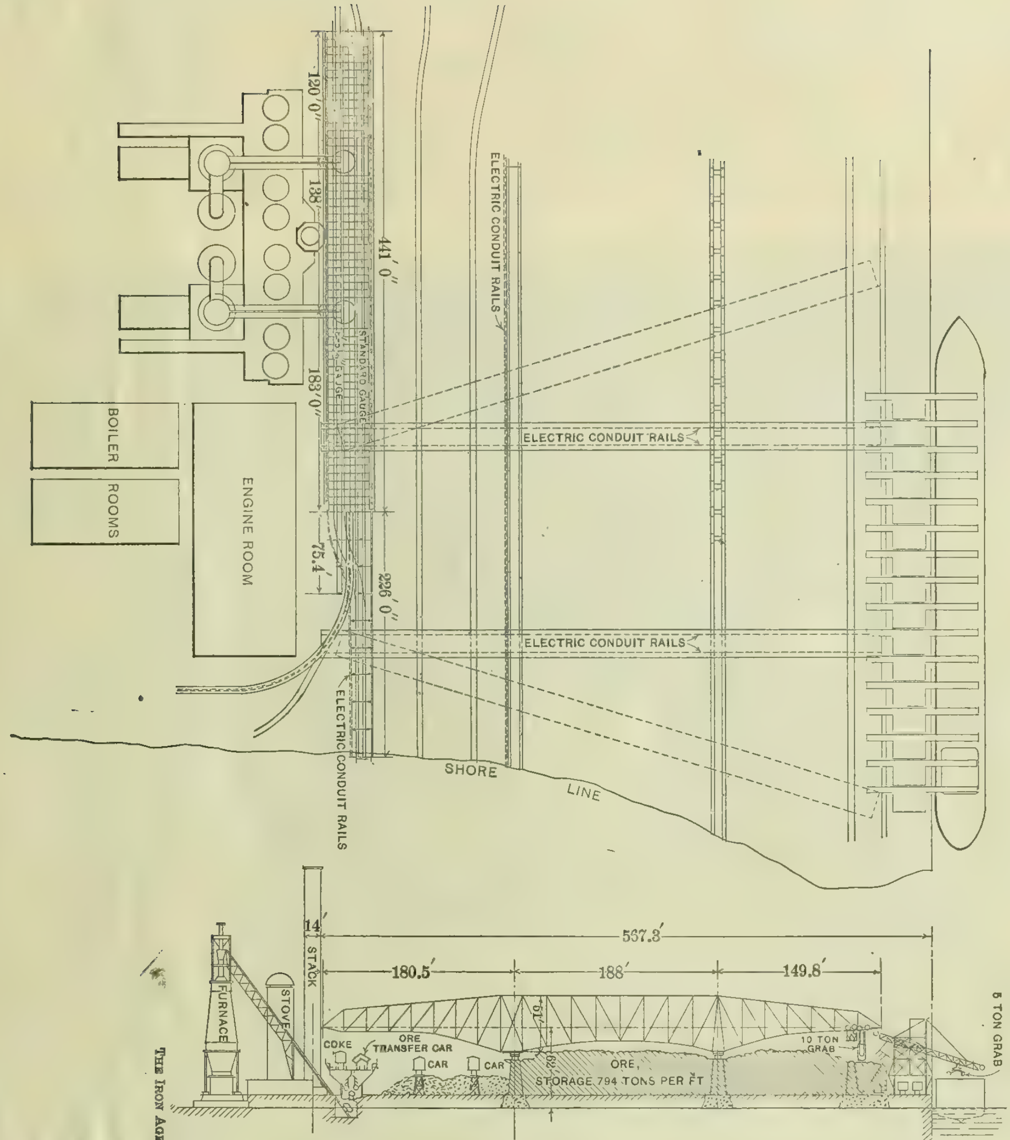


Fig. 1. General Plan and Elevation.

Fig. 2. Electric Larry or Scale Car under Pockets.

Art. 2. The Unloaders.

The unloaders are seventy-foot towers about thirty by forty feet. They span two standard railroad tracks, and can be moved along the dock on their own tracks, under their own power. Each tower supports an inclined pivoted arm or boom that carries a five-ton grab. The object of the pivot is that the arm may be raised to clear the vessel moving in the slip. See Fig. 3. The grab can be lowered from any point along the arm and can be dumped automatically either into the weighing hopper in the lower part of the tower, or into the trough at the rear of the tower under the other end of the arm. See Fig. 1 and 3.

The unloaders average 125 to 200 tons per hour per machine at an average cost of about three cents per ton.

The power used for the unloaders is steam, taken by flexible connections from a steam main that runs the full length of the dock front, in a six-foot tunnel that is in the wall

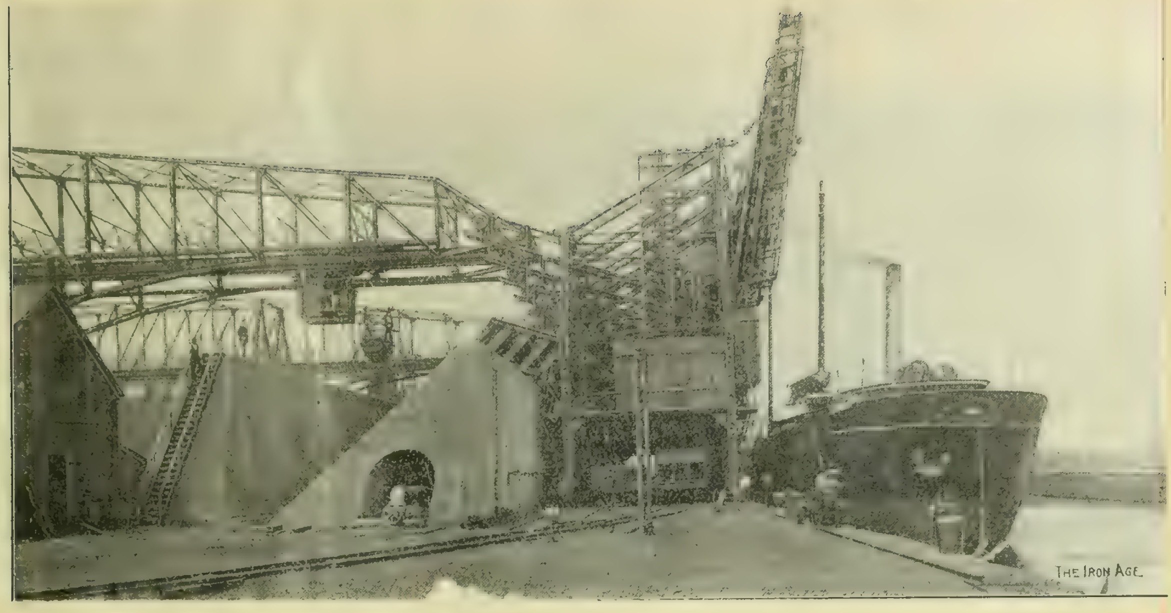
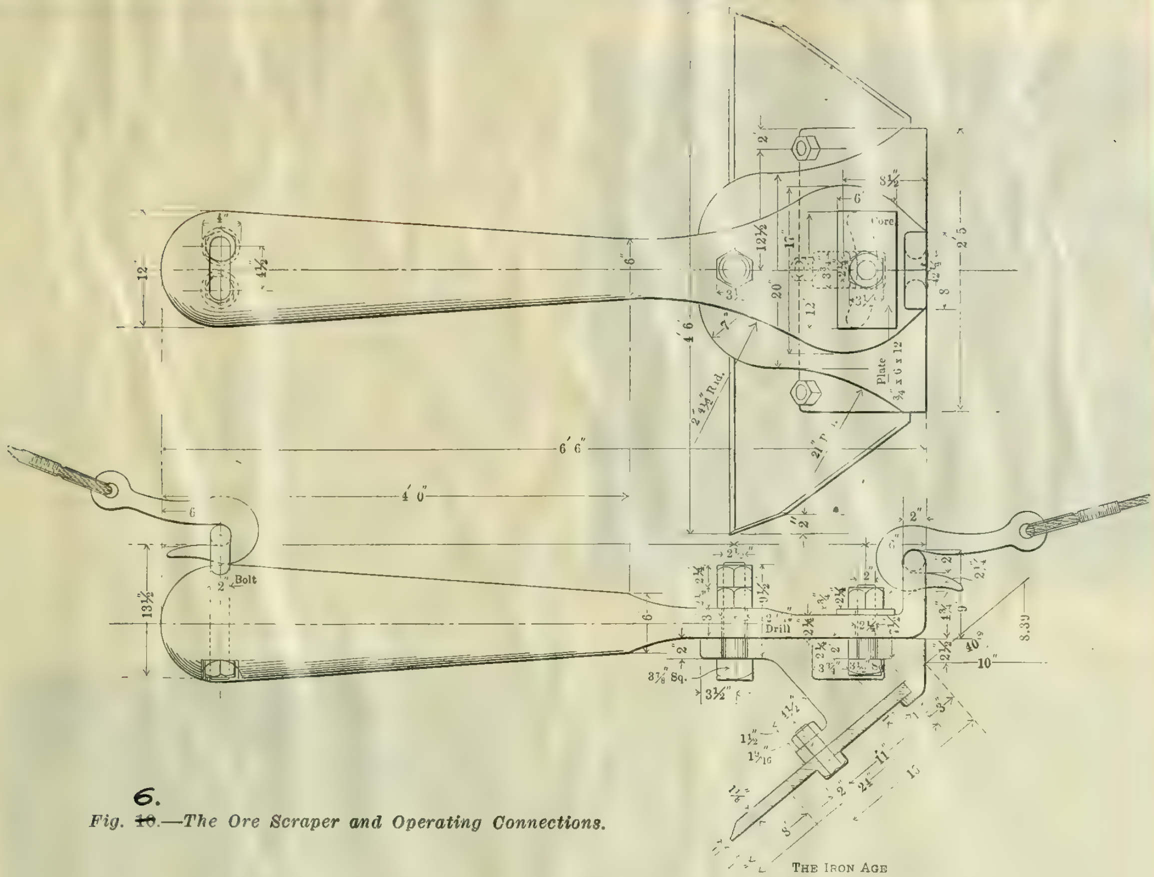


Fig. 3. End View of Battery of Unloaders.
Concrete Receiving Trough.

Fig. 4. Unloaders in Position for Work.
Fig. 5. General View of Plant. (over)

of the V-shaped trough. See Fig. 3. This tunnel is also used as passageway. Each unloader requires one man to operate it. This man also has charge of the engine running the scraper. that is used in the hold of the vessel for pulling the ore up to the hatch where the grab can get it. This scraper is nothing more or less than an enormous hoe, with a blade four and a half feet wide. Its operation is controlled by a workman in the hold. See Fig. 6 and 7. By its use, the manual labor that was formerly necessary is done away with, and much time is saved.

One of the early records made possible by the help of this scraper, was the removal of one entire hatch of 540 tons of ore, in four hours and one minute at a cost of two dollars per hour, or less than one and one half cents per ton. This included scraping about one half of it to the hatch.



6.
Fig. ~~10~~.—The Ore Scraper and Operating Connections.

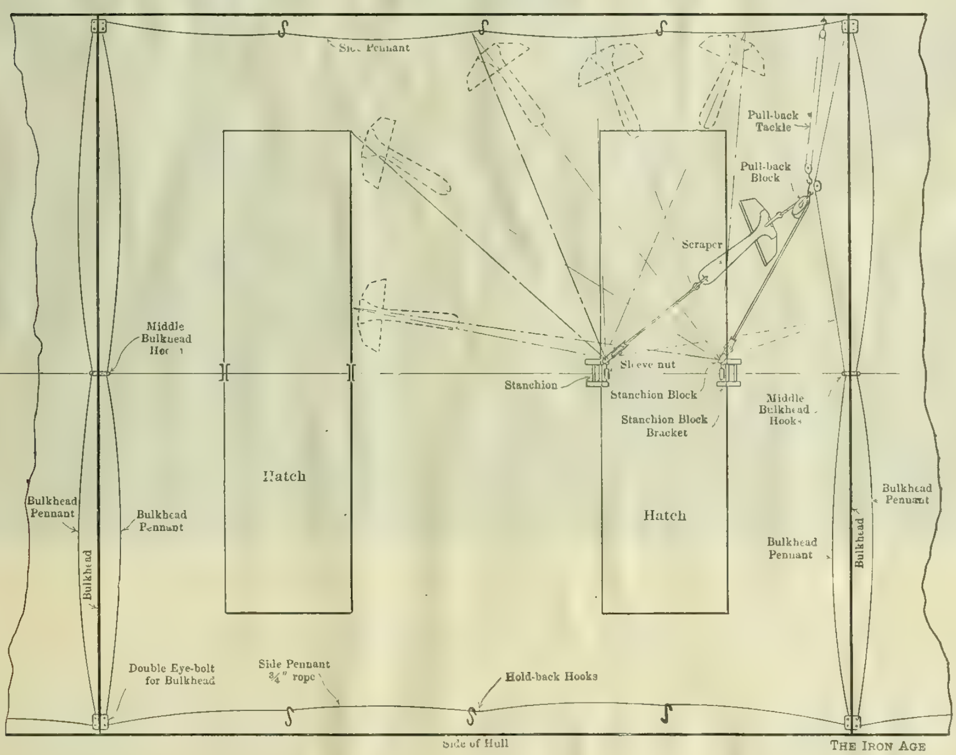


Fig. 7. Showing Operation of Scraper.

Art. 3. The Receiving Trough.

The receiving trough is made for the most part of slag-concrete; the first portion built was of brick. It is V-shaped being about 16 ft deep and 20 ft. across the top, and has a round bottom to facilitate the work of the grabs in taking out the ore. The back-face of the "V" is vertical, while the one on the side of the unloaders is at an angle of about 60° with the vertical. Under the latter face is the tunnel already mentioned, which extends the full length of the dock; see Fig. 3.

Art. 4. The Receiving Tracks.

As shown in Fig. 3 and also at extreme right hand side of Fig. 1, the unloaders span two standard tracks. These tracks are for the purpose of giving to furnaces, other than those served by the plant directly, the benefit of cheap, quick unloading; then too, probably, some time is saved in the unloading, as the

grabs would have a slightly shorter trip than when they run out to the end over the receiving trough.

Art. 5. The Traveling Gantry Cranes.

The traveling gantry-cranes, or ore-bridges as they are more commonly known, are about 520 ft. long, and are supported by two concrete walls or piers, 188 ft. center to center, that are parallel to the dock as shown on Fig. 1. and 8. These piers serve as dividing lines for the different grades of ore in the yard, as well as runways for the ore bridges. From Fig. 1. it is seen that each ore-bridge has a central span of 188 ft. and two over-hanging or cantilever arms of about 180 and 150 ft. respectively.

Over each pier is a tower that is supported on a car. The car on the north pier has a bearing between the two trusses that acts as a pivot; in addition to this, is a set of cone bearings. The tower on this car is called the fixed tower and that on the south pier is called the

roller tower because of the double set of rollers that permits both lateral and transverse motions of the bridge on the car itself. The object of this is, that the bridge may be skewed to a position other than normal to the piers. The maximum skew is 17° either way from the normal; the total distance between the two maxima is about 115 ft. measured along the pier, if the fixed tower is not moved. The end of the furnace arm moves over 220 ft. between the two maximum skewed positions. Fig. 1. shows the bridge in both the normal and a skewed positions. This property of the bridge is more valuable than would at first be supposed, as it gives a much better command of the storage yard, and makes possible a better discrimination in the storing of different grades or varieties of ore.

The bridge is operated electrically. For the movement of the bridge along the piers there are two 35 HP Westinghouse railway motors and for operating the

grab there are two 125 HP motors of the same type. The maximum capacity of the grab is about 13 tons, but the average load is about 10 tons. The maximum transverse speed of the grab is about 1000 ft. and the maximum hoisting speed about 100 ft. per minute. The bucket itself weighs 11 tons and the entire trolley when loaded weighs something over 100,000 lb.

The operator of the bridge and his assistant travel with the load, and have complete control of all movements of bridge and trolley so that it is possible to load or unload with exactness from any point on the bridge. The grab may deliver to cars on trestles over the reserve stock piles or directly into the transfer cars on the ore pockets as well as any point of the yard. See Fig. 8.

Fig. 8. Traveling Gantry Cranes.

Ore Pockets.

Battery of Unloaders

Skip-Hoist.

Storage Yard.

Reserve Limestone and Coke.

Stoves.

Art. 6. The Ore Storage Yard.

As already stated and as shown by Fig. 1 and 8, the supporting piers form division walls for different grades of ore. The space between the receiving trough and the south pier is used entirely for ore storage, the capacity being 794 tons per linear foot. Between the south pier and the ore pockets, are the reserve stock-piles of coke and limestone which, in addition to the ore bridges are served by tracks on trestles.

Art. 7. The Ore Pockets.

The ore pockets are huge steel troughs which are divided into compartments of seventy tons capacity. The compartments are in pairs, and as shown on Fig. 9, the series on the yard side are for ore and those on the furnace side are for coke and limestone. The pockets are supported on two rows of steel columns which also carry a standard gage and a wide-gage track over each series of pockets. The wide gage is for the transfer cars, one of which is shown

in Fig. 9. These cars are $\wedge$ bottomed with hinged sides that are used to distribute the ore or other material along the pockets as it is received from the grab of the ore bridge.

The pockets are of a general V shape longitudinally, with vertical ends. At the bottom of the V shown on Fig. 2 and 9, is a most ingenious roller screen, which has for its most important function, the measuring out of a definite amount of material. By throwing in a gear the roller is connected to a shaft and caused to revolve, and by revolving a certain distance, a certain amount of material is passed out and caught on the electric laury or scale car. See Fig. 2.

Each of the six furnaces has its own laury and pair of skip hoists. The capacity of the laury is that of two skip hoists or about 40,000 lb. Another ingenious thing is the automatic recorder on the laury which weighs every bit of material taken onto the car. This recorder is correct to 0.3 of one per cent. By the use of the electric laury, the recorder,

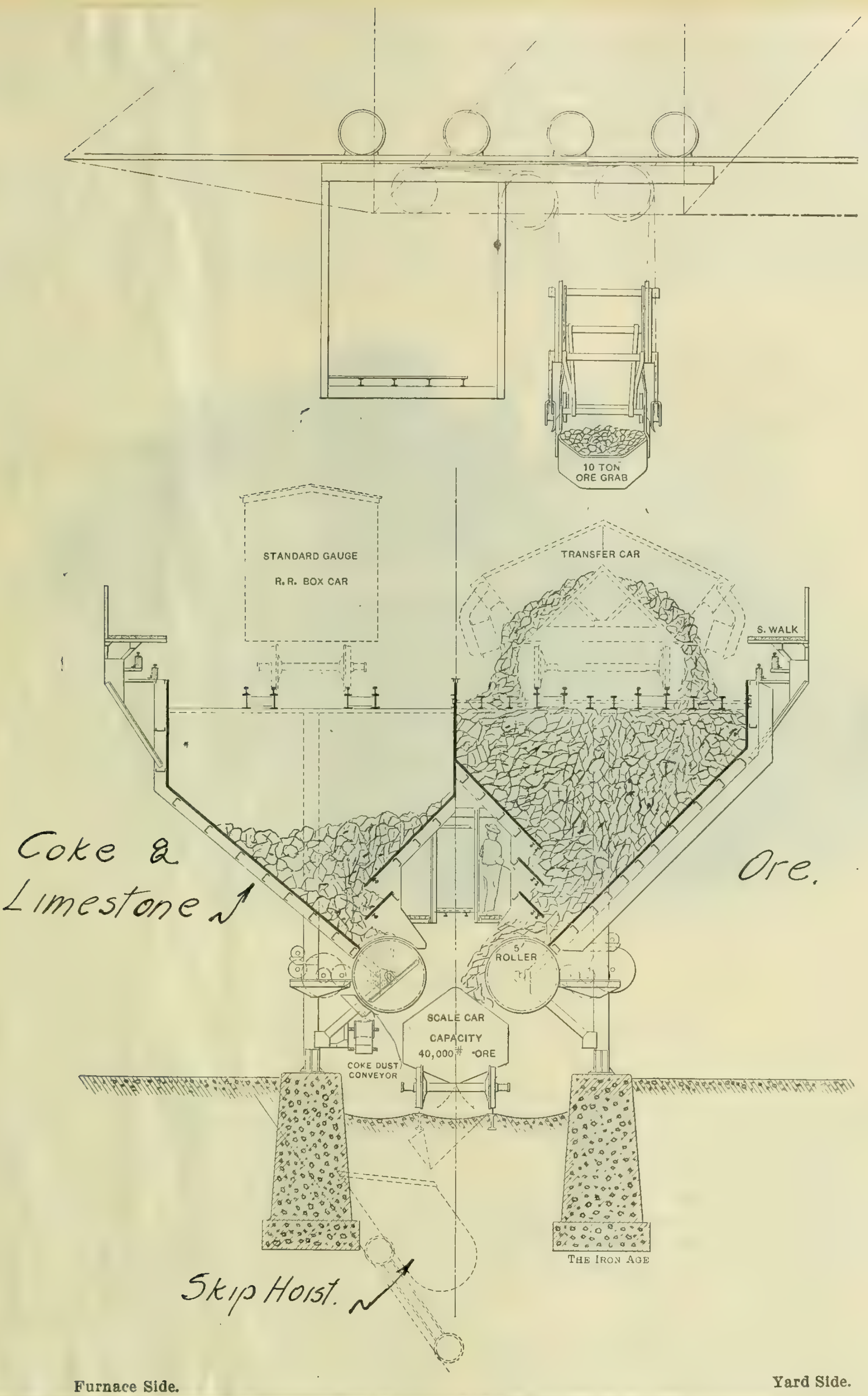


Fig. 9. Cross Section of Pockets, Lawry and Hoist.
Fig. 10. The electric Lawry or Scale Car.

and the skip-hoists only one man is required to supply each furnace.

To avoid the freezing of the ore, or to thaw out already frozen ore, hot air is circulated through the double walls of the pockets.

Art. 8. The Skip-Hoists.

The skip-hoist is the modern furnace-charging machine. It has superseded the old wheelbarrow elevator at all up to date plants. The essentials are an inclined track running from beneath the electric lawry to the top of the furnace stack, and a car which, when pulled to the top by a cable, will automatically discharge its contents into the charging bells. The skip-hoists in this plant are in pairs, each having a capacity of about ten tons. The movement of the skip hoists is controlled by the operator of the lawry. See Fig. 9 and 10.

Part II.

Investigation of the Gantry Crane.

Art. 9. General Form and Dimensions.

The gantry crane consists of two Pratt trusses 518.5 ft. in length, spaced 25 ft. center to center and supported, as already mentioned, on piers that are 188 ft. center to center; see Fig. 1. The cantilever arms are 149.9 ft. and 180.6 ft. long on the dock and furnace ends respectively. The central span is 181.5 ft., there being eight panels, each of which is 22'-8¼". The upper chord of this span is straight but the lower is curved except in the two center panels. The cantilever arm on the dock end has seven panels of 21'-5" making a length of 149'-11" or 149.9'. The upper chord of this arm is sloped down from the pier panel or fixed tower, where the truss is 51' deep to the end vertical post where the truss is 18' deep; the bottom chord is curved; see Fig. 1.

The cantilever arm on the furnace end has the same general shape, but is longer, and has the

three end panels with straight lower chords. There are nine panels including the pier panel or roller tower, of 20'-9½" each, thus giving a total length of 187'-1½"

In each of the towers the lower chords consist of heavy box-girders which are cross-braced and thoroughly tied together. The box girders of the fixed tower are 17'-8½" long and 5'-0" deep; those of the roller tower are of the same depth but are 27'-0" long. In both towers, the girders extend several feet beyond the panel points.

The supporting cars under the towers are 60 ft long and consist briefly of two heavy girders, 4 ft. deep, thoroughly cross braced, that rest on 16 pairs of wheel-trucks of 6'-4" gage. The actual wheel base is 47'-4", the extensions beyond trucks on each end being used for brakes.

Art. 10. Computation of the Dead Load.

The following computations were made for the purpose of comparing the actual dead load and the assumed dead loads that were used in the computation of stresses. To make it easier to distinguish between the various members, each panel was considered by itself; the weight of each part of each member was computed separately. It was thought that, because of the great similarity in the panels, that the weights of some could be obtained by a ratio between two panels of computed weight. This ratio method was adopted in several cases. The weight of details was assumed as a per cent of the weight of the main members.

Dead Load.

Weight of One Truss.-

Member	Composition	Wgt. per ft.	Length	Weight	Total
L_0U_1	2[s 12×20.5	20.5	26	1,060	
L_0L_1	2[s "	"	22	900	
	Details 25%			500	
Panel #2.				2,460	2,500
U_1L_1	2[s 12×20.5	20.5	18	740	
U_1U_2	2[s "	"	21.5	880	
U_2L_1	4Ls $5\times3\frac{1}{2}\times\frac{1}{2}$	13.6	30	1,630	
L_1L_2	2[s 12×25	25.	21.5	1,080	
	Details 25%			4330 1,080	5,400
Panel #5 U_4L_4	2[s 12×20.5	20.5	27.5	1,130	
U_4U_5	2[s 15×33	33.	22.	1,450	
	1 c. pl. 19×$\frac{1}{2}$	32.3	22.	710	
U_5L_4	4Ls $5\times3\frac{1}{2}\times\frac{7}{16}$	12.	36.5	1,750	
L_4L_5	2[s 15×45	45	22.	1,980	
	1 c.pl. 19×$\frac{1}{2}$	32.3	22	710	
	Details 25%			7,730 1,930	
				9,660	9,700
	By ratio	#2 to	#5		
	Panel #3				6,800
	" #4				8,300
	" #6				11,200

Panel # 7

$U_6 L_6$	2 [s 12 × 20.5	20.5	42.5	1,740	
$U_6 L_7$	2 [s 15 × 40	40.	22	1,760	
	1 c.pl. 19. × $\frac{5}{8}$	40.4	22	880	
$U_7 L_6$	4 Ls 5 × $3\frac{1}{2}$ × $\frac{3}{8}$	10.4	50.	2080	
	Details	25%		6460 1,615	8,100
#8 $U_7 U_8$	2 [s 15 × 50	50	22.7	2,270	
	1 c.pl. 19 × $\frac{1}{2}$	32.3	..	730	
$U_7 L_8$	2 [s 12 × 30	30	51	3,060	
$L_{7b} L_8$	2 [s 15 × 50	50	17.5	1,750	
	1 c.pl. 19 × $\frac{5}{8}$	40.4	17.5	710	
	Details	25%		8,520 2,130	10,600
#12. $U_{11} U_{12}$	2 [s 15 × 40	40.	22.7	1,810	
$U_{11} L_{11}$	2 [s 12 × 20.5	20.5	42.	1,720	
$U_{12} L_{11}$	2 [s 12 × 30	30	48	2,890	
$L_{11} L_{12}$	2 [s 15 × 40	40	22.7	1,810	
	1 c.pl. 19 × 5/8	40.4	22.7	920	
	Details	25%		9,150 2,290	11,400
	By ratio				
Panel	# 9				11,500
	10				11,500
	11				11,500
	13				11,800
	14				13,700
	15				15,500

Panel #16 Roller Tower –

$U_{15} L_{15}$	2 [s 15 × 40	40	51	4,080	
$U_{15} U_{16}$	2 [s 15 × 50	50	43.5	4,350	
	2 side pls. 12 × 7/16	17.9	42	1,500	
	1 c. pl. 19 × 3/8	24.3	43.5	1,050	
$U_{16} L_{15}$	2 [s 12 × 30	30	50	3,000	
$L_{16} U_{15}$	2 [s 12 × 25	25	50	2,500	
$L_{15} L_{16}$	1 Pl 20 × 5/8	42.5	24	1,020	
Box G. Girder	2 Pl. 60 × 1/2	102	27	5,500	
	2 Ls 6 × 6 × 5/8	24.2	27	1,310	
	2 Ls 6 × 4 × 5/8	20	27	1,080	
	1 Pl. 24 × 1 3/4	142.8	27	3,860	
	2 [s 15 × 50	50	23	2,300	
	2 Pl. 15 × 9/16	28.7	24	1,380	
	Details	20 %		35,090 7,010	42,100
#17. $U_{16} L_{16}$	2 [s 15 × 40	40	51	4,080	
$U_{16} U_{17}$	2 [s 15 × 50	50	20.8	2,080	
	2 pls. 12 × 5/16	12.7	20.8	530	
	1 c. pl. 19 × 7/16	28.3	20.8	590	
$U_{17} L_{17}$	2 [s 12 × 20.5	20.5	23.	940	
$U_{16} L_{17}$	4 Ls 5 × 3 1/2 × 7/16	12.	48	2,300	
$L_{16} L_{17}$	2 [s 15 × 45	45	22	1,980	
	2 spl. 12 × 7/16	17.8	22	790	
	1 c. pl. 19 × 5/8	40.4	22	890	
	Details 25 %			14,180 3,540 17,720	17,700
					209,300

Panel #18			Brought up		209,300
$L_{17}L_{18}$	2 [s 15×45	45	21.5	1,930	
	2 pl. 12×$\frac{5}{16}$	12.7	21.5	550	
	1 c.pl. 19×5/8	40.4	21.5	870	
$L_{18}U_{17}$	4 Ls 5×3$\frac{1}{2}$×$\frac{1}{2}$	13.6	43.	2,340	
$L_{18}U_{18}$	2 [s 12×20.5	20.5	35.5	1,450	
$U_{17}U_{18}$	2 [s 15×50	50	21.	2,100	
	1 pl. 19×$\frac{1}{2}$	32.3	21	680	
	Details 25%			9,920 2,500	12,400
#21. $L_{20}L_{21}$	2 [s 15×33	33	21	1,390	
	1 c.pl. 19×$\frac{9}{16}$	36.3	21	760	
$L_{21}U_{20}$	4 Ls 5×3$\frac{1}{2}$×$\frac{1}{2}$	13.6	32.5	1,720	
$L_{21}U_{21}$	2 [s 12×20.5	20.5	23.5	970	
$U_{20}U_{21}$	2 [s 12×25	25	21.	1,050	
	2 s. pls. 9×$\frac{7}{16}$	13.4	21.	560	
	Details 25%			6,450 1,610	8,100
	By ratio				
	Panel #19				10,900
	" 20				9,500
	" 22				7,300
	" 23				5,000
	" 24				2,500

Total for One truss 265,000.

Top Lateral System.

Continuous Strut

4 Ls	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	5.0#	476'	9,500.	9,500

Top Laterals

8 Ls	$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5.6		1,480	
8 Ls	$3 \times 3 \times \frac{3}{8}$	7.2		1,900	
16 Ls	$4 \times 3 \times \frac{7}{16}$	9.8	33'	5,180	
12 Ls	$3 \times 3 \times \frac{5}{16}$	6.1		2,420	
6 Ls	$4 \times 3 \times \frac{3}{8}$	8.5		1,680	
16 Ls	$4 \times 3 \times \frac{1}{2}$	11.1		5,860	
4 Ls	$4 \times 3 \times \frac{5}{16}$	7.7		950	19,170

Intermediate Struts.

84 Ls	$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5.6	23.5'	11,050	11,050

End Struts

8 Ls	$6 \times 4 \times \frac{3}{8}$	12.3	23.5	2,310	
2 Ls	$4 \times 4 \times \frac{5}{16}$	8.2	41.	670	
Details 18%				530	2,810
Total - Top Laterals					42,530

Sway Bracing.

36 Ls	$3 \times 3 \times \frac{5}{16}$	6.1#	total	934.'	5,700
8 Ls	$6 \times 4 \times \frac{5}{8}$	20.	"	188.	3,760
3 Ls	$3 \times 3 \times \frac{5}{16}$	6.1	"	39.	640
8 Ls	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	5.	"	58.	290
8 Ls	$6 \times 4 \times \frac{5}{8}$	20.	"	187.	3740
					13,730

Top Lat. + Sw. Br = 56,260

Brought forward 56,260

Floor Beams (20)

2 L^s	6 x 4 x $\frac{1}{2}$	16.2#	23.4'	760	
4 L^s	3 x 3 x $\frac{5}{16}$	6.1	7.6	190	
2 L^s	3 x 3 x $\frac{5}{16}$	6.1	6.2	80	
8 L^s	5 x 3$\frac{1}{2}$ x $\frac{3}{8}$	10.4	8.8	730	
2 L^s	6 x 4 x $\frac{7}{16}$	14.3	23.4	670	
Details 15%				360	
20 Fl. B. @				2,790	55,800

4 Special Fl. B.

2 L^s	6 x 4 x $\frac{1}{2}$	16.2	23.4	760	
4 L^s	6 x 4 x $\frac{3}{8}$	12.3	23.2	1,150	
4 L^s	5 x 3$\frac{1}{2}$ x $\frac{1}{2}$	13.6	6.1	330	
8 L^s	5 x 3$\frac{1}{2}$ x $\frac{3}{8}$	10.4	8.2	680	
4 L^s	3 x 3 x $\frac{5}{16}$	6.2	7.3	180	
Details 15%				460	
4 Fl. B @				3,560	14,240

Hangers (48.)

4 L^s	5 x 3$\frac{1}{2}$ x $\frac{1}{2}$	13.6#	7.2'	390	
2 L^s	4 x 4 x $\frac{5}{16}$	8.2	8.5	140	
4 L^s	3 x 3 x $\frac{5}{16}$	6.1	6.5	160	
Details 15%				100	
48 Hangers @				790 =	37,900

Total carried forward
(exclusive of truss.) 164,200

For both trusses 164,200

Stringers (44)

2 Ls $5 \times 3\frac{1}{2} \times \frac{9}{16}$	15.2#	22.6	690	
2 Ls "	15.2	21.8	670.	
1 pl. $32 \times \frac{3}{8}$	40.8	22.6	930	
4 Ls $5 \times 3\frac{1}{2} \times \frac{3}{8}$	10.4	2.4	100	
40 St. @			2,390 =	95,600
4 specials - @			2,000 =	8,000
2 " @			670 =	1,340

2 End Stringers

4 Ls $6 \times 4 \times \frac{3}{8}$	12.3#	25.	1230	
1 pl. $48 \times \frac{3}{8}$	61.2	26.7	1630	
2 E. S. @			2860	5,720

Stringer Bracing.

30 Ls $4 \times 4 \times \frac{5}{16}$	8.2#	13.2	3,250	
8 Ls "	"	12.0	750	
30 Ls "	"	13.8	3,400	
22 Ls "	"	13.3	2,400	
6 Ls "	"	11.4	560	
Details 15%			1550	11,910

Total for both trusses 286,770

or carried by one truss 143,380

Addenda (One truss)

60# rails - $\frac{518.5}{3}$ (on stringer) 10,370

80# " " (conductors) 13,800

Extra - one truss 167,550

			Extras.		167,550
Girder under Fixed Tower					8,590
Outside Bracing in Towers-					
R.T. 1 pl.	$14\frac{1}{2} \times \frac{5}{16}$	15.4#	52'	800	
2 Ls	$6 \times 4 \times \frac{3}{8}$	12.3	53	1,560	
2 Ls	$3 \times 3 \times \frac{5}{16}$	6.1	46.5	570	
Details 15%				440	3,370
Girder Struts.					
1 pl.	$23 \times \frac{3}{8}$	29.2	23.2	680	
2 Ls	15 × 33	33.	23.9	1,580	
Details 15%				340	2,600 one tr. 1,300
Cross Frames					
2 Ls	$4 \times 3 \times \frac{5}{16}$	7.2	77.6×2	550	
Details 10%				50	600
F.T. Outside Bracing. (each side)					
1 pl.	$14\frac{1}{2} \times \frac{5}{16}$	15.4	20.4	310	
1 pl.	"	"	25.5	390	
2 Ls	$6 \times 4 \times \frac{3}{8}$	12.3	23.2	580	
2 Ls	"	"	26.5	660	
2 Ls	$3 \times 3 \times \frac{5}{16}$	6.1	25.2	310	
Details 10%				220	2,470
Girder Struts					
1 pl.	$23 \times \frac{3}{8}$	29.2	23.3	680	
2 Ls	15 × 33	33	23.9	1,580	
1 L	15 × 40	40	10.7	450	
Details 15%				410	
				3,120 one tr.	1,560
					185,440

	185,440
Hand Railing and Walk Stringers	12,400
Continuous Strut (between posts)	6,810
Total load due to weight of bridge, on each truss,	204,650
Weight of truss	265,000
Total weight carried by each truss	469,650 lb.

Art. 11. Comparison with Assumed Dead Loads.

The assumed dead loads per truss were as follows: at L_0 and L_{24} 8,000; L_{1-2} and L_{23-24} 12,000; L_{3-4-5} and $L_{19-20-21}$ 16,000; L_6 and L_8 to 14, L_{17} and L_{18}, 20,000; at L_{7a} and L_{7b} 30,000; L_{15} and L_{16} 35,000 lb. The total of the assumed loading is 490,000 while the total of the computed weight is 469,650 lb., a difference of 20,350 lb. in favor of the assumed loading which goes to increase the factor of safety.

Art. 12. The Conditions for maximum and minimum stresses.

The maximum stress in any member is the sum of the dead, live and wind stresses. The wind load is considered only when it exceeds 30% of the dead plus the live loads, and then it is reduced $\frac{1}{3}$ as the allowable wind stress is 150% of the allowable dead and live load stresses.

The wind load is assumed as twenty lb. per sq. ft. of exposed surface of both trusses. The area of the stringers is counted only once as they are pretty close together. There is also a wind load assumed as 6,000 lb. acting on the trolley, which is treated as a live load.

The minimum stresses will be the dead load stresses alone or the sum of dead, live and wind loads as above indicated.

The position of the bridge considerably affects the intensity of the stresses. The following diagram shows

the changes in the lengths of the arms and span caused by moving from the normal to the maximum skewed position.

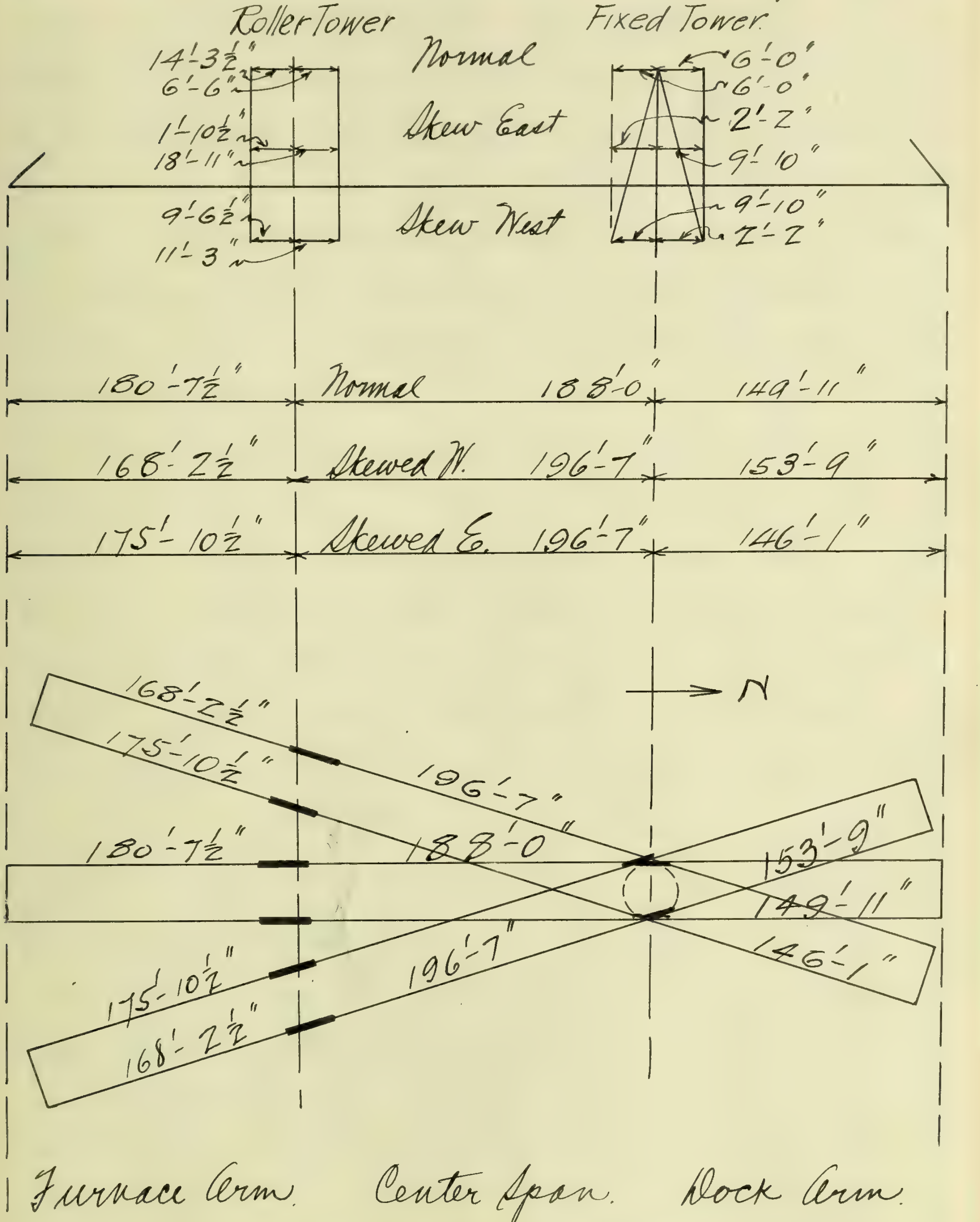


Fig. 11.

See also Fig. 1.

Art. 13. The Dead Load Stresses.

The dead load stresses of the dock arm were calculated by moments. In these computations, the sign of the moments is considered as positive when counter-clockwise. For the stresses, plus indicates compression and minus, tension. In taking the moments, the moment arms were scaled directly from the drawings.

A comparison between the stresses as found for dead load and the minimum stresses given on the stress sheet, shows the stresses as here computed to check very closely in most cases and exactly in some.

	kips
$-21 \times 8 - 15 L_0U_1 = 0$	$L_0U_1 = -11.2$
$+17 L_0L_1 - 21 \times 8 = 0$	$L_0L_1 = +9.5$
$+42 \times 8 - 63 U_1L_1 = 0$	$U_1L_1 = +5.3$
$-18 U_1U_2 - 21 \times 8 = 0$	$U_1U_2 = -9.3$
$+20.5 L_1L_2 - 43 \times 8 - 21 \times 12 = 0$	$L_1L_2 = +28.8$
$+110 L_1U_2 + 160 \times 12 + 138 \times 8 = 0$	$L_1U_2 = -27.4$
$-181 U_2L_2 + 160 \times 12 + 138 \times 8 = 0$	$U_2L_2 = +16.7$
$-20.5 U_2U_3 - 21 \times 12 - 43 \times 8 = 0$	$U_2U_3 = -29.$
$+23 L_2L_3 - 21 \times 12 - 43 \times 12 - 64 \times 8 = 0$	$L_2L_3 = +55.5$

(Note - The kip as used here represents 1,000 lb.)

$-202\, U_3L_3 + 138\times 8 + 160\times 12 + 181\times 12 = 0$ $\qquad U_3L_3 = +25.8$

$+94\, U_4L_3 + 113\times 16 + (92+72)12 + 50\times 8 = 0$ $\qquad U_4L_3 = -44.5$

$-23\, U_3U_4 - (21+43)12 - 64\times 8 = 0$ $\qquad U_3U_4 = -56.$

$+27.5\, L_3L_4 - 21\times 16 - (43+64)12 - 85\times 8 = 0$ $\qquad L_3L_4 = +84.$

$-134\, U_4L_4 + 114\times 16 + (92+72)12 + 50\times 8 = 0$ $\qquad U_4L_4 = +31.$

$-27\, U_4U_5 - 21\times 16 - (43+64)12 - 85\times 8 = 0$ $\qquad U_4U_5 = -85.3$

$+83\, L_4U_5 + (90+69)16 + (48+27)12 + 8\times 7 = 0$ $\qquad L_4U_5 = -42.$

$+33\, L_4L_5 - (21+43)16 - (64+85)12 - 106\times 8 = 0$ $\qquad L_4L_5 = +111.1$

$-111\, U_5L_5 + (90+69)16 + (48+27)12 + 7\times 8 = 0$ $\qquad U_4L_5 = +31.4$

$-33.5\, U_5U_6 - (21+43)16 - (64+85)12 - 106\times 8 = 0$ $\qquad U_5U_6 = -111.1$

$+78\, L_5U_6 + (82+61+40)16 + (18-4)12 - 25\times 8 = 0$ $\qquad L_5U_6 = -34.8$

$+41\, L_5L_6 - (21+43+64)16 + (85+106)12 + 127\times 8 = 0$ $\qquad L_5L_6 = +131.$

$-106\, U_6L_6 + (82+61+40)16 + (18-4)12 - 25\times 8 = 0$ $\qquad U_6L_6 = +25.6$

$-42\, U_6U_7 - (21+43+64)16 + (85+106)12 + 127\times 8 = 0$ $\qquad U_6U_7 = -128.$

$+50\, L_6L_{7a} - 21\times 20 - (43+64+85)16 - (106+127)12$

$\qquad - 140\times 8 = 0$ $\qquad L_6L_{7a} = +150.$

$+87\, L_6L_7 + 86\times 20 + (65+44+22)16 + 1\times 12$

$\qquad - 20\times 12 - 41\times 8 = 0$ $\qquad L_6U_7 = -37.5$

As a typical computation of the dead load stresses, the member $U_{10}-U_{11}-U_{12}$ at the middle of the center span, will be considered. The condition for maximum dead load stress is, that the bridge be in the normal position.

To treat the loads more easily, they will be divided into the three groups

on the two arms and span respectively and the distance of the center of gravity of each group, from the extreme north end determined. For the dock arm, the equation of moments about the extreme end is as follows: (the unit of loads is 1,000 lb. and the unit distance is one panel length of 21'-5.") k = kip = 1,000 lb.

$0 \times 8k + 1 \times 12 + 2 \times 12 + (3+4+5)16 + 6 \times 20 + 6.75 \times 30 = xp \times 130k$, 130 being the sum of the loads on the arm. The total moment thus found is 550 k.p., which, when divided by 130k, gives 4.24 p or $4.24 \times 21'$-5" = 91' from the end to the center of gravity of loads on the dock arm. For the center span, with the center of moments under the first load considered (near the fixed tower) which is 156 ft. from the end, the equation is as follows: $0 \times 30k + (1+2+3+4+5+6+7)20 + 8 \times 35 = xp \times 205k = 840k$

xp is then $\frac{840k}{205k} = 4.1$ and since each panel is 22'-8$\frac{1}{4}$", 4.1p = 93'-0" from the center of moments or 93 + 156 = 249 ft. from the extreme north end of the bridge.

For the furnace arm, with the center of moments under first load to left of the roller tower (352.2' from end) the equation is $0 \times 35 + (1+2)20 + (3+4+5)16 + (7+8)12 + 9 \times 8 = 155 \times p k = 514 k$, which upon being solved gives 69' from the center of moments or $69 + 352.2 = 421.2'$ from the extreme north end. The loads may then be concentrated as follows:

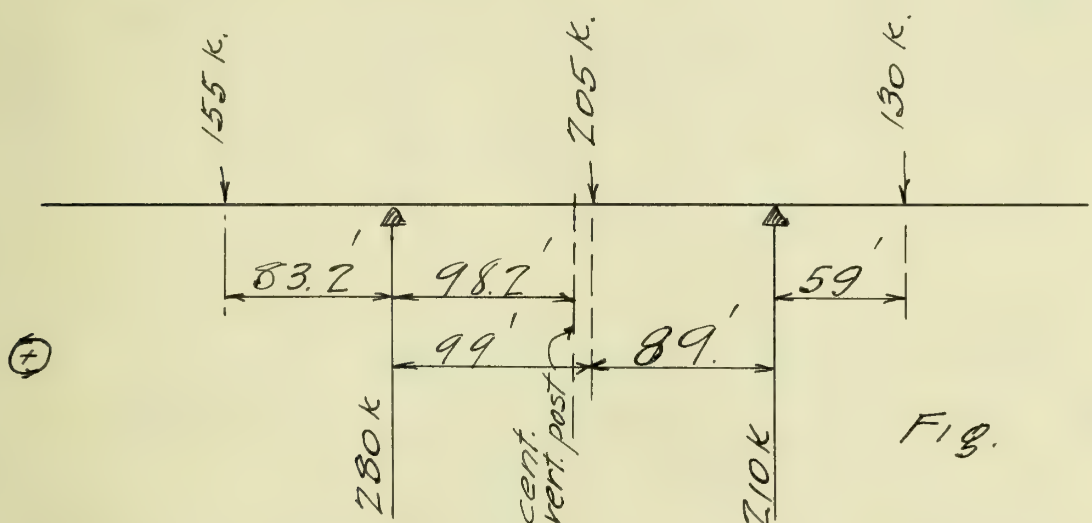


Fig.

The computation of the reactions is as follows - center of moments at right reaction - $+(89+99+83.2) \times 155 - (89+99)R_1 + 89 \times 205 - 59 \times 130 = 0$ From which R_1 is found to be 280 k ; R_2 is then $490 - 280 = 210$ k.

Note - The point marked cent. vert. post is the post at the center of the member being considered, $U_{10}-U_{11}-U_{12}$.

The bending moment at the center vertical post is equal to the resisting moment in the member or the stress in the member times the depth of the truss at that point. The bending moment is + (98.2 + 83.2) × 155 − 98.2 × 280 − 51.2 × 115 which equals − 42.5 S. Solving, S is found to be − 152 kips or 152,000 lb. tension. The value 115 used in the equation is the sum of loads on center span to left of vertical post and 51.2′ is the distance to center of gravity.

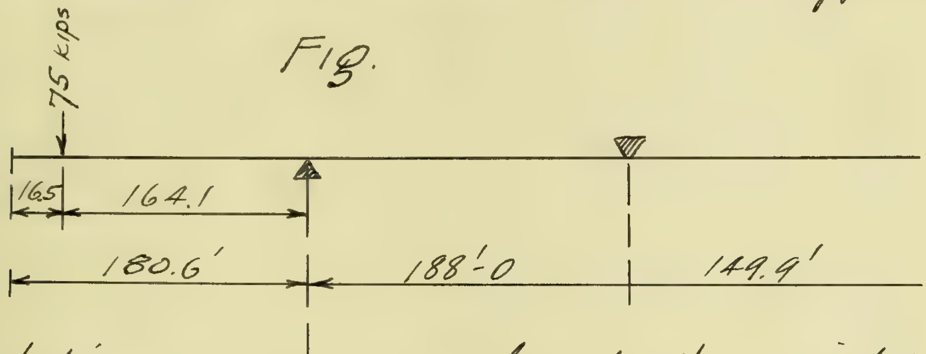
Art. 14. The Live Load Stresses.

The live load consists of the trolley which is supported on four axles, seven feet apart. The actual weight of the trolley when fully loaded, is between fifty and sixty tons, but it is assumed to weigh seventy-five tons, which means that 75,000 lb. is to be carried by each truss or 18,750 lb. per wheel. The trolley can travel to a point six feet from either end of the bridge.

For the maximum live load stress

in the upper chord, the trolley must be at the end of the longest cantilever arm, when the bridge is in normal position. The member $U_{10}-U_{11}-U_{12}$ will be considered and the computation of the live load stress will be shown as typical.

Fig.



By taking moments about the right support the equation is $-188\,R_1 + 352.1 \times 75\,k = 0$ from which $R_1 = +141.\,k$ and since the load is $75\,k$, $R_2 = 75 - 141 = -66$ kips. The distance 16.5 ft. is the distance from the end of the bridge to center of gravity of the trolley.

The bending moment at the center vertical post is $+262.3 \times 75 - 98.2 \times 141$ which equals $42.5\,S$, or $+6{,}300, = -42.5\,S$; S is therefore $-\frac{6{,}300,\cancel{000}}{42.5} = -148\,k$ or 148,000 lb. tension.

Part III.

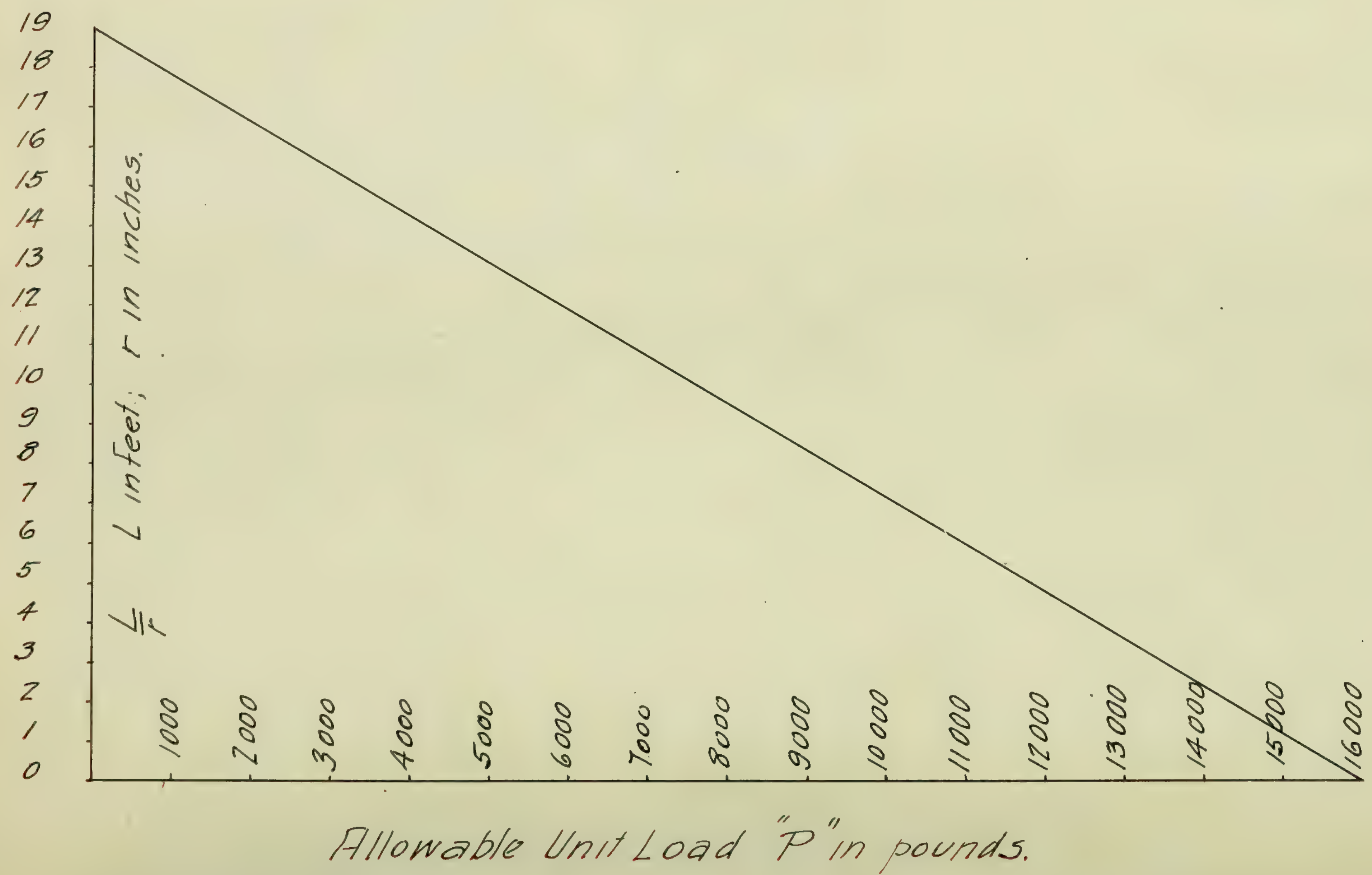
Investigation of Members.

In the investigation of members of the truss, the stresses used in the computations are those computed by the designers from the assumed loadings. The allowable unit stress in tension is 16,000 ~~lbs~~ per sq. in and that in compression is $16,000 - 70\frac{l}{r}$. For the purpose of simplifying the computations, a curve will be drawn for the equation $P = 16,000 - 70\frac{l}{r}$, from which the values of P for any value of $\frac{l}{r}$ may be taken directly. see Pl. I.

The values of rivets will be taken according to the specifications for the bridge, as 7,500 lb. per sq. in. for field, 9,000 for shop rivets in shear, and 15,000 and 18,000 lb. per sq. in. for field and shop rivets in bearing, respectively. The shearing and bearing values will be given for reference in Table I.

PLATE I.

CURVE SHOWING VALUES OF $P = 16{,}000 - 840\frac{L}{r}$.



M. E. DEP'T

UNIVERSITY OF ILLINOIS.

MACHINE DESIGN.

Subject:—

Drawing No. .... Date .... By ....

Table I.
Rivet Values.

Bearing	3/4" rivet.		7/8" rivet.	
in	Shop	Field.	Shop.	Field.
3/8" plate	5,070	4,230	5,920	4,940
7/16	5,900	4,910	6,880	5,740
1/2	6,750	5,620	7,880	6,570
9/16	7,600	6,340	8,800	7,340
5/8	8,400	7,000	9,850	8,210
11/16	9,300	7,750	10,800	9,000
3/4	10,100	8,400	11,800	9,850
Shear.	3,980	3,320	5,400	4,500

Art. 15. Tension Members - Top Chord.

A few of the segments of the top chord will be worked out in full and results for the remainder tabulated.

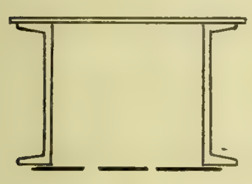
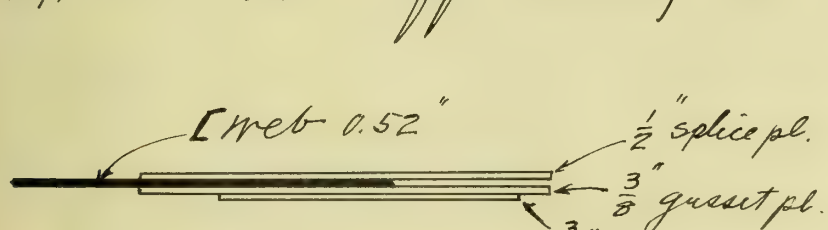


Fig. 12.

$U_9 - U_{10}$ is composed of 2 [s 15" × 40 lb. and 1 cover plate 19" × $\frac{1}{2}$", the under side being laced. The total stress carried is 325,000 lb. which requires an area of $\frac{325{,}000}{16{,}000} = 20.3$ □". The gross areas are, Cover plate 9.5 □" and [s 23.52 □". In the cover plate there are 4 $\frac{3}{4}$" rivets and in the [s there are 6 - $\frac{7}{8}$" rivets in the webs and 2 $\frac{3}{4}$" rivets in the flanges in one section, which gives net sections as follows - cover plate 7.87 □" and [s 19.53 □" or a total for the member of 27.4 □". The efficiency is therefore $\frac{27.4}{20.3} = 135\%$.



Channel Connection

Fig. 13.

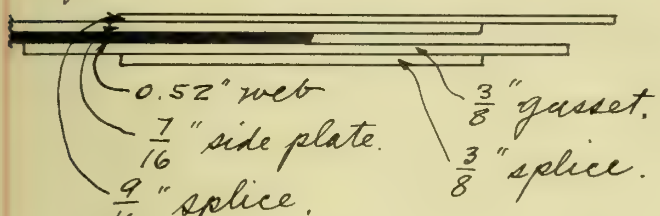
Cover Pl. Splice

As shown in Fig. 13, the cover-plate is spliced by a $\frac{7}{16}$" plate. There ar 20 $\frac{3}{4}$" shop rivets in the connection. The stress carried by the cover-plate is $\frac{7.87}{27.4} \times 325{,}000 = 93{,}500$ lb. which would require $\frac{93{,}500}{3{,}980} = 24$ rivets. The efficiency is $\frac{20}{24} = 83\%$. Each of the channels is connected to a $\frac{1}{2}$" plate by 15 $\frac{7}{8}$" shop rivets in its web. The stress carried by each channel and plate is $\frac{325{,}000 - 93{,}500}{2} = 115{,}750$ lb. which is divided between them. The plate carries $\frac{0.50}{0.50 + 0.52} \times 115{,}750 = 56{,}800$ lb. which would require $\frac{56{,}800}{5400} = 11$ $\frac{7}{8}$" shop rivets. The efficiency is therefore $\frac{15}{11} = 136\%$.

<u>U_{14}-U_{15}-U_{16}.</u> This member has a similar composition with the addition of two side plates to the composition of the member U_9-U_{10}. See Fig. 12. The maximum stress carried is 482,000 lb. which requires an area of $\frac{482{,}000}{16{,}000} = 30.1$ □". The gross area of the cover plate is 7.13 □" and the net is 5.91 □". The gross area of the Is is 29.42 □" while the net is 25.37 □", and the net area of the two side plates (12" × $\frac{7}{16}$") is 7.88 □"

giving a total net area of $39.16\,\square''$ and an efficiency of $\frac{39.16}{30.1} = 130\%$.

The cover plate stress of $\frac{5.91}{39.16} \times 482{,}000$ or 72,300 lb is transferred by 21 $\frac{3}{4}''$ shop rivets to a $\frac{1}{2}''$ splice plate. The required number of $\frac{3}{4}''$ shop rivets is $\frac{72{,}300}{5{,}070} = 15$ and the efficiency is $\frac{21}{15} = 140\%$.



Channel Connection
Fig. 14.

The connection with the next segment is shown in Fig. 14 and consists of the following members or parts on each side: the channel whose web is 0.52" thick, the side plate $\frac{7}{16}''$ thick, a $\frac{9}{16}''$ splice plate outside and a $\frac{3}{8}''$ splice on the inside besides the $\frac{3}{8}''$ gusset. The stress carried by the half shown, is $\frac{482{,}000 - 72{,}300}{2} = 204{,}850$ lb. which is divided up as follows: The total thickness is 2.28" and therefore the $\frac{9}{16}''$ splice carries $\frac{0.56}{2.28} \times 204{,}850 = 50{,}400$ lb. the $\frac{3}{8}''$ gusset and $\frac{3}{8}''$ splice each carry $\frac{0.38}{2.28} \times 204{,}850 = 34{,}200$ lb.

The $\frac{9}{16}''$ splice-plate carries 50,400 lb. by means of 17 $\frac{7}{8}''$ shop rivets, while the required

number is only $\frac{50,400}{5,400} = 10$, being governed by the shearing value. The $\frac{3}{8}$" splice-plate has 9 $\frac{7}{8}$" shop rivets to carry its stress 34,200 lb.; the required number is $\frac{34,200}{5,400} = 7$. The $\frac{3}{8}$" gusset plate carries its own load of 34,200 lb. in addition to that of the $\frac{3}{8}$" splice plate, 34,200 lb. or 68,400 in all, by means of 17 rivets. The 9 rivets thru the $\frac{3}{8}$" splice also go thru the $\frac{3}{8}$" gusset, but 7 of them are required for the splice which leaves a balance of 10 for the $\frac{3}{8}$" gusset-plate. The required number is $\frac{68,400}{5,920} = 12$ and consequently the efficiency is $\frac{10}{12} = 83\%$.

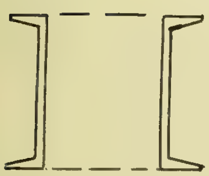


Fig. 15.

End Post L_0U_1. This post is composed of 2 [s 12" × 25 lb. laced on both top and bottom. The maximum direct stress that it carries is 48,000 lb. which would require an area of $\frac{48,000}{16,000} = 3.0$ □". The net area is 9.96 □". There is however a considerable bending moment for this member to carry. The 6,000 lb wind load is partly transferred to the end and divided between

the trusses. The post is 26' long; the panel 20.8' and the 6,000 lb load at a point 16.5' from the end. The part of the load causing a bending moment is then $\frac{20.8 - 16.5}{20.8} \times \frac{6,000}{2} =$ 620, say 650 lb. The connection of the portal is 14' from the lower end of the post and the bending moment is therefore 650 × 14 × 12 = 109,000."# Since the member is in tension the formula, for stress due to bending moment, is $S = \dfrac{My_1}{I + \frac{Pl^2}{10E}}$ in which M = bending moment, y_1 is the distance from neutral axis to extreme fiber, P = total direct stress, and l = length in inches. Substituting in this formula S is found to be:

$$S = \frac{109,000 \times 6}{256.2 + \dfrac{48,000 \times (316)^2}{10 \times 28,000,000}} = 2,400 \text{ lb. per sq. in.}$$

The direct stress is $\frac{48,000}{9.96} = 4,820$ lb. per sq. in so that the total is 2,400 + 4820 = 7,220. Since the allowable is 16,000 lb. per sq. in., the efficiency is $\frac{16,000}{7,220} = 221\%$.

The channels are fastened at the bottom by 7 $\frac{7}{8}$" field rivets to a $\frac{3}{8}$" plate. The stress carried by each channel is $\frac{48,000}{2} = 24,000$ lb.

which requires $\frac{24,000}{3,630}$ = 7 rivets; the efficiency of the connection is $\frac{7}{7}$ = 100%

The efficiency of the connection to the top chord is $\frac{8}{7}$ = 114%, since the connection is again governed by the bearing of the $\frac{7}{8}$" field rivet in the 0.28" web of the channel, and 8 rivets were used.

The remaining segments of the top chord will be investigated for section area only, as the set of drawings available is not complete and does not contain their details. In order to condense the investigation, the results will be tabulated. The method followed is the same as heretofore used, viz. the total stress divided by the allowable unit stress gives the required area and the given net section divided by the required net section gives the efficiency of the member.

Top Chord Segments.

Mark	Stress	Section	Area Net.	Allowable Unit Stress	Required Net Area	Efficiency of member.
$U_1 U_2$	41,000	2 [s 12 × 20.5	10.32	16,000	2.56	402 %
$U_2 U_3$	133,000	2 [s 12 × 25	12.96	"	8.3	155 %
$U_3 U_4$	228,000	2 [s 15 × 33 1 pl. 19 × $\frac{5}{16}$	22.7	"	14.25	159.
$U_4 U_5$	297,000	2 [s 15 × 33 1 pl. 19 × $\frac{1}{2}$	25.29	"	18.6	136.
$U_5 U_6$	330,000	2 [s 15 × 40 1 pl. 19 × $\frac{1}{2}$	29.51	"	20.6	143.
$U_6 U_7$	367,000	2 [s 15 × 40 1 pl. 19 × $\frac{5}{8}$	31.56	"	22.9	137.
$U_7 U_8$	400,000	2 [s 15 × 50 1 pl. 19 × $\frac{1}{2}$	34.70	"	25.0	139.
$U_8 U_9$	362,000	2 [s 15 × 50 1 pl. 19 × $\frac{3}{8}$	32.68	"	22.6	144
$U_{10} U_{11} U_{12}$	308,000	2 [s 15 × 40 1 pl. 19 × $\frac{7}{16}$	28.47	"	19.4	146
$U_{12} U_{13}$	362,000	2 [s 15 × 45 1 pl. 19 × $\frac{1}{2}$	32.19	"	22.6	142
$U_{13} U_{14}$	421,000	2 [s 15 × 45 2 pls. 12 × $\frac{5}{16}$ 1 pl. 19 × $\frac{3}{8}$	36.4	"	36.3	100.
$U_{16} U_{17}$	453,000	2 [s 15 × 50 2 pls 12 × $\frac{5}{16}$ 1 pl. 19 × $\frac{7}{16}$	39.95	"	38.3	104.

$U_{17}U_{18}$	403,000	2[s 15×50 1 pl. 19×½	34.73	16,000	35.2	98.7%
$U_{18}U_{19}$	350,000	2[s 15×40 1 pl. 19×⅝	31.56	"	21.9	144.
$U_{19}U_{20}$	289,000	2[s 15×40 1 pl. 19×5/16	26.45	"	18.1	146.
$U_{20}U_{21}$	213,000	2[s 12×25 1 pl. 9×7/16	19.08	"	13.3	143
$U_{21}U_{22}$	117,000	2[s 12×25	12.96	"	7.3	177
$U_{22}U_{23}$	31,000	2[s 12×20.5	10.32	"	1.94	530.
$U_{23}L_{24}$	41,000	2[s 12×20.5	10.11	"		

Bending Moment = 109,000 inch lb.

Unit Stress Bend. Moment = 2,420 lb. per sq. in.

" " Direct St. = $\frac{41,000}{10.11}$ = 4,050 " " "

6,470 " " "

Efficiency = $\frac{16,000}{6,470}$ = 247%

Art. 16. Tension Members - Diagonals.

The method of investigation of the diagonals is the same as that used for the top chord segments. As a typical computation the member $L_8 U_7$ will be worked out in full.

Fig. 16.

Like all the other diagonals of the center span, this member is composed of 2 [s 12" x 30 lb. laced on both sides. It is subject to a reversal of stress and consequently must be examined in both tension and compression. The stresses are + 63,000 lb. and − 108,000 lb. The member is 51' long and its radius of gyration is 4.28" which gives a value of 6,800 lb. from the curve on Pl. I. The required area is $\frac{63,000}{6,800} = 9.27$ □" for compression and since the gross area is 17.64 □" the efficiency in compression is $\frac{17.64}{9.27} = 190\%$. The tensile stress 108,000 lb. requires a net area of $\frac{108,000}{16,000} = 6.75$ □"; the given net area is 14.58 □" and the efficiency in tension is therefore, $\frac{14.58}{6.75} = 216\%$.

The stress to be used in the design of rivets and splices, according to the

specifications is $108,000 + \frac{3}{4}(63,000) = 155,200$ lb. or 77,600 lb. for each channel. The connection is made by 18 $\frac{7}{8}$" field rivets directly thru the web of the channel and a $\frac{3}{8}$" gusset-plate. The required number is governed by the bearing in a $\frac{3}{8}$" plate and is $\frac{77,600}{4,940} = 16$, giving an efficiency of $\frac{18}{16} = 113\%$.

$L_{14}U_{15}$ - This is another diagonal that is subjected to a reversal of stress, the stresses being $-121,000$ and $+31,000$ lb. The member is 51.' long and has a radius of gyration of 4.78, consequently the value of P is 6,800 lb. and the required area for compression is $\frac{31,000}{6,800} = 4.56$ □". The gross area is 17.64 □" and the efficiency in compression, therefore, is $\frac{17.64}{4.56} = 386\%$. In tension, the required area is $\frac{121,000}{16,000} = 7.55$ □" and the given net area is 15.60 □" or an efficiency of $\frac{15.60}{7.55} = 207\%$ for the member.
The connection to each chord is made by 32 $\frac{7}{8}$" field rivets to $\frac{3}{8}$" gusset plates. The stress for the computation of rivets is $121,000 + \frac{3}{4}(31,000)$ or 144,200 lb. The required number of rivets is $\frac{144,200}{4,940} = 30$ and the efficiency is $\frac{32}{30} = 107\%$.

$L_{16}U_{15}$ - This member is the cross-bracing of the roller tower and is strictly a tension member. It is composed of 2 [s 12" × 25 lb. with a net area of 12.36 □" The maximum stress computed for this member is 110,000 lb. which requires an area of $\frac{110,000}{16,000} = 6.9$ □" net. The efficiency is, therefore, $\frac{12.36}{6.9} = 179\%$

The connections are made by 16 $\frac{7}{8}$" field rivets to $\frac{3}{8}$" gusset plates on the chord segments. The required number is governed by the bearing value of a $\frac{7}{8}$" field rivet on a $\frac{3}{8}$" plate and is $\frac{110,000}{4,940} = 22$. The efficiency of the connection is then, $\frac{16}{22} = 73\%$.

The investigation of the remaining diagonals will be confined to an examination of the sectional areas, for the reasons given on p. 52, under the subject of top chord segments. The method used is the same as was used in that article and in this article.

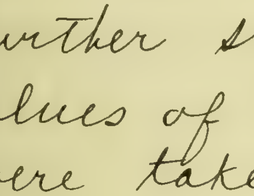
Diagonals.

Mark	Stress	Section	Net Area	Allowable Unit Stress	Required Area	Efficiency.
$L_1 U_2$	133,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{1}{2}$	14.0	16,000	8.3	168.%
$L_2 U_3$	146,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{9}{16}$	15.6	"	9.2	170.
$L_3 U_4$	135,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{1}{2}$	14.0	"	8.4	167.
$L_4 U_5$	118,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{7}{16}$	12.4	"	7.4	168.
$L_5 U_6$	105,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{3}{8}$	10.7	"	6.6	162.
$L_6 U_7$	107,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{3}{8}$	10.7	"	6.7	160.
$L_9 U_8$	108,000	2 [s 12×30	15.4	"	6.7	230.
$L_{10} U_9$	90,000	2 [s 12×30	15.4	"	5.6	275.
$L_{11} U_{12}$	114,000	"	"	"	7.2	214.
$L_{12} U_{13}$	118,000	"	"	"	7.4	208.
$L_{13} U_{14}$	125,000	"	"	"	7.8	198.
$L_{15} U_{16}$	138,000	"	"	"	8.7	184.
$L_{17} U_{16}$	116,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{7}{16}$	12.4	"	7.3	170.
$L_{18} U_{17}$	130,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{1}{2}$	14.0	"	8.1	175.
$L_{19} U_{18}$	137,000	"	"	"	8.6	163.
$L_{20} U_{19}$	140,000	"	"	"	8.8	160.
$L_{21} U_{20}$	141,000	"	"	"	8.8	160.
$L_{22} U_{21}$	147,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{9}{16}$	15.6	"	9.2	169.
$L_{23} U_{22}$	127,000	4 Ls $5 \times 3\frac{1}{2} \times \frac{7}{16}$	12.4	"	8.0	155.

Art. 17. Compression Members.

As was stated on p. 44 in a general discussion of the investigation of members, the values for the allowable unit stresses in the compression members, will be taken from the curve $P = 16,000 - 70\frac{l}{r}$, plotted on p. 45 To still further simplify the computations the values of "r," the radius of gyration were taken from the handbook known as "Godfrey's Tables."

The investigation of a few members will be given in full and the results for the remainder will be tabulated.



$\underline{L_0 L_1}$. The member L_0L_1 is 21'-8⅜" long and is subjected to a compressive strength of 42,000 lb. as a maximum. The two channels composing the member are 11½" b-b which is considerably more than the distance b-b to make the radii of gyration equal, so it is safe to say that the radius about the axis perpendicular to the web of channel, is the least.

From the tables this "r" is found to be 4.61 which makes $\frac{L}{r} = 4.71$ and $P = 12,300$ lb. The required area is $\frac{42,000}{12,300} = 3.41$ □" and the efficiency is $\frac{12.06}{3.41} = 355\%$. This excessive efficiency is explained by the fact altho the channels used in this member were the minimum used on the bridge, they were still, larger than necessary.

The connection at L_1 is made by a $\frac{5}{16}$" plate in which there are 4 $\frac{3}{4}$" shop rivets, 2 $\frac{3}{4}$" field rivets and 10 $\frac{7}{8}$" field rivets in the webs of the channels. The bearing on the webs will be the controlling factor for determining the number of rivets. The value of the connection is as follows: $10 \times \frac{7}{8} \times 0.28 \times 15,000 = 36,800$ lb. for the 10 $\frac{7}{8}$" rivets and $4 \times \frac{3}{4} \times 0.31 \times 18,000 + 2 \times \frac{3}{4} \times 0.31 \times 15,000 = 16,700 + 6,970$ or 23,670 lb. for the 6 $\frac{3}{4}$" rivets, making a total of 60,470 lb. This is amply sufficient as only 42,000 lb. was required. The efficiency of the connection is therefore $\frac{60,470}{42,000} = 143\%$. The connection at the other end, L_0, is by 10 $\frac{7}{8}$" rivets (shop) thru the channel webs and $\frac{3}{8}$" plates. The total bearing value of these rivets is $10 \times \frac{7}{8} \times 0.28 \times 18,000 = 44,100$ lb. or an efficiency of $\frac{44,100}{42,000} = 105\%$.

$L_6 L_{7a}$ This member is composed of 2 [s 15"×50 lb. and a cover plate 19"×½" which is on the lower side. It is 16'-5½" long, and has a radius of gyration of 5.5" which gives $\frac{L}{r}$ the value of 30, and P = 13,500 lb. per sq. in. The total stress 375,000 lb. requires $\frac{375,000}{13,500}$ = 27.8□" while the given area is 39.2□"; the efficiency is therefore $\frac{39.2}{27.8}$ = 141%.

Fig. 16.

The connection at L_6 is made by 7 ⅞" shop rivets in a ½" plate on one side and a ⅜" gusset-plate on the other side of each web and 16 ¾" shop rivets in a ½" splice plate on the cover plate. The value of the connection is as follows: the cover plate stress is $\frac{19\times\frac{1}{2}}{39.2}$ × 375,000 = 93,000 lb. which, if transferred by ¾" shop rivets would require $\frac{93,000}{3,980}$ = 24; the efficiency is, then, $\frac{16}{24}$ = 67%. The stress in the channels is 375,000 − 93,000 = 282,000 lb. for which there are 14 ⅞" shop rivets and 6 ⅞" field rivets all in double shear. Their total value in bearing is 14 × ⅞ × 0.72 × 18,000 + 6 × ⅞ × 0.72 × 15,000 = 158,500 + 56,800 = 215,300 lb, while the

total required is 282,000 lb. so that the efficiency is $\frac{215,300}{282,000} = 76\%$. The connection is apparently below the requirements if all stress is to be transferred by the splice plates. This, however, is not the case, as the ends of the abutting members of the joint are milled, and it is therefore all-right to consider that some of the stress is transferred directly from one member to the other.

The other end is connected directly to the $\frac{7}{16}$" webs of the box-girder by 48 $\frac{7}{8}$" field rivets through the 0.72" Channel-web and the $\frac{7}{16}$" girder-web and 30 $\frac{7}{8}$" field rivets through $\frac{3}{8}$" angles to the $\frac{7}{16}$" girder-web. The value of the connection is $48 \times 4,500 + 30 \times 4,500 = 352,000$ lb. and the efficiency therefore is $\frac{352,000}{375,000} = 94\%$.

$L_{7b}\,L_8$. See Fig. 16. This member is very similar to the one just investigated, the only differences being in the total stress carried, which is 396,000 lb. and the size of the cover plate. The upper side is laced. $L_{7b}\,L_8$ is 16'-6½" long; its area

is 41.30 □" and its radius of gyration is 5.66". The value of $\frac{L}{r}$ is 2,92 and of P is 13,400 lb. The required area is therefore $\frac{396,000}{13,400} = 29.5$ □" which makes the efficiency $\frac{41.3}{29.5} = 140\%$

In the connection at L_{7b} to the box-girder, there are 52 $\frac{7}{8}$" field rivets directly through the 0.72" webs of the channels and the $\frac{7}{16}$" webs of the box-girder, and 32 $\frac{7}{8}$" field rivets through the $\frac{3}{8}$" angles to the $\frac{7}{16}$" webs. The total value of the connection is (52 + 32) 4500 = 378,000 lb. The efficiency is $\frac{378,000}{396,000} = 96\%$.

The connection at L_8 is by 24 $\frac{7}{8}$" field rivets through the 0.72" webs of the channels and $\frac{3}{8}$" gusset-plates and $\frac{1}{2}$" splice plates. The cover plate splice has 40 $\frac{3}{4}$" field rivets; the cover plate itself is $\frac{5}{8}$" thick while the splice is $\frac{1}{2}$" thick. The value of the connection is $24 \times \frac{7}{8} \times 0.72 \times 15,000 + 40 \times 3320 = 133,000 + 227,000 = 360,000$ lb. The efficiency is $\frac{360,000}{396,000} = 91\%$ if all stress is transferred through the splice, but as stated for the member $L_6 L_{7b}$, it is fair to assume that some of the stress is transferred directly through the milled ends and, therefore, the connection is efficient.

$L_{12}L_{13}$ This member has the same general shape as that given in Fig. 16. The two Channels are 15" × 40 lb and the bottom cover plate is 19" × $\frac{5}{8}$"; the top is laced. The stress carried as a maximum is 393,000 lb. The length is 22'-8 $\frac{9}{16}$" (both ends are milled), and its radius of gyration is 5.76, so that $\frac{L}{r}$ is $\frac{22.7}{5.76} = 3.94$ and the value of P is 12,800 lb. The required area is $\frac{393,000}{12,800}$ = 30.7□" and the given area is 35.4□" so that the efficiency is $\frac{35.4}{30.7} = 115\%$.

See Fig. 16.

The connection at L_{12} has 14 $\frac{7}{8}$" shop rivets and 6 $\frac{7}{8}$" field rivets through the channel web, a $\frac{3}{8}$" gusset-plate and a $\frac{1}{2}$" splice-plate on each side and 16 $\frac{3}{4}$" shop rivets through the $\frac{5}{8}$" cover-plate and a $\frac{1}{2}$" splice-cover-plate. The value of the connection is $14 \times \frac{7}{8} \times 0.52 \times 18,000 + 16 \times 3,980 + 6 \times \frac{7}{8}'' \times 0.52 \times 15,000 = 219,800$ lb. and the efficiency is $\frac{219,800}{393,000} = 56\%$, showing that the joint was probably designed to transfer part of the stress directly by means of the milled ends.

At L_{13} there are 20 $\frac{3}{4}$" field rivets in the cover-plate splice, and 24 $\frac{7}{8}$" field rivets

through the channel web, the $\frac{3}{8}$" gusset-plate and $\frac{1}{2}$" splice plate. The total value of these rivets is $20 \times 3{,}980 + 24 \times \frac{7}{8} \times 0.57 \times 15{,}000 = 247{,}600$ lb. giving an efficiency of $\frac{242{,}600}{393{,}000} = 62\%$, but some of the stress will go through the milled ends and consequently, the joint may be considered efficient.

The remaining segments of the lower chord, with the exception of the box-girders under the towers, will be investigated for the section area only. The box-girders mentioned will be taken up in Art. 19.

The method of procedure for the investigation of the remaining segments of the lower chord, is the same as that already used in this article, viz; the allowable unit stress is governed by the ratio $\frac{L}{r}$; the total stress in the member, divided by this unit allowable stress gives the required area and the given area divided by the required area gives the efficiency of the member.

Efficiencies of Lower Chord Members.

Mark	Stress	Section	Length	Rad. of Gyration	$\frac{L}{r}$	Allowable Unit Stress	Given Area	Required Area	Efficiency
L_1L_2	133,000	2[s 12×25	21.4'	4.43	4.84	12,000	14.7	11.1	132%.
L_2L_3	226,000	2[s 12×25 2 s. pl: 9"×$\frac{9}{16}$	21.4 ~~21.5~~	3.8 ~~5.7~~	5.64	11,300	24.8	20.0	124%
L_3L_4	296,000	2[s 15×40 1 pl. 19×$\frac{3}{8}$	21.5	5.7	3.77	12,900	30.6	23.0	133.
L_4L_5	337,000	2[s 15×45 1 pl. 19×$\frac{1}{2}$	22.0	5.72	3.85	12,800	35.9	26.3	136.
L_5L_6	360,000	2[s 15×50 1 pl. 19×$\frac{7}{16}$	22.0	5.60	3.94	12,700	37.7	28.3	130
L_8L_9	372,000	2[s 15×50 1 pl. 19×$\frac{9}{16}$	22.9	5.66	4.05	12,700	39.2	29.3	134
L_9L_{10}	336,000	2[s 15×40 1 pl 19×$\frac{5}{8}$	22.8	5.76	3.96	12,700	35.4	26.4	134
$L_{10}L_{11}$	293,000	2[s 15×33 1 pl. 19×$\frac{9}{16}$	22.7	5.85	3.88	12,700	30.8	22.9	134
$L_{11}L_{12}$	337,000	2[s 15×40 1 pl. 19×$\frac{5}{8}$	22.7	5.76	3.94	12,700	35.5	26.3	135.
$L_{13}L_{14}$	448,000	2[s 15×50 2 s.pl. 12×$\frac{5}{16}$ 1 c.pl. 19×$\frac{9}{16}$	22.8	5.78	3.95	12,700	47.2	35.4	133.
$L_{14}L_{15}$	486,000	2[s 15×50 2 s. pl. 12×$\frac{7}{16}$ 1 c.pl. 19×$\frac{5}{8}$	23.0	5.65	4.07	12,700	51.2	38.2	134.
$L_{16}L_{17}$	459,000	2[s 15×45 2 s.pls. 12×$\frac{7}{16}$ 1 c.pl. 19×$\frac{5}{8}$	22.0	5.6	3.93	12,700	48.8	36.2	135.

$L_{17}L_{18}$	435,000	2 [s 15×45 2 s. pls. 12×$\frac{5}{16}$ 1 pl. 19×$\frac{5}{8}$	21.0	5.77	3.64	12,900	45.8	33.7	136 %
$L_{18}L_{19}$	400,000	2 [s 15×40 2 s.pl. 12×$\frac{5}{16}$ 1 c. pl. 19×$\frac{5}{8}$	20.9	5.70	3.67	12,900	42.9	31.0	138.
$L_{19}L_{20}$	352,000	2 [s 15×45 1 c. pl. 19×$\frac{9}{16}$	20.9	5.71	3.66	13,000	37.2	27.1	137.
$L_{20}L_{21}$	289,000	2 [s 15×33 1 c. pl. 19×$\frac{9}{16}$	20.9	5.85	3.57	13,100	30.5	22.0	139.
$L_{21}L_{22}$	213,000	2 [s 12×25 2 side pls 9×$\frac{1}{2}$	20.8	3.80	5.50	11,400	23.7	18.7	127.
$L_{22}L_{23}$	117,000	2 [s 12×25	20.8	4.4	4.73	12,000	14.7	9.78	150.
$L_{23}L_{24}$	32,000	2 [s 12×20.5	21.0	4.61	4.55	12,300	12.0	2.6	460.

Art. 18. Compression Members - Vertical Posts

The vertical posts, (except in towers) are all composed of two 12 inch channels, of various weights according to stresses. They are laced on both sides. As a typical computation the member U_5L_5 will be investigated as to area and connections and the others will be investigated as to area only and the results tabulated.

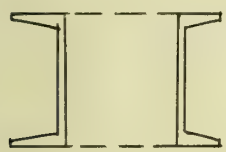


Fig. 17.

$\underline{U_5L_5}$. This member is $34'-0\frac{1}{16}''$ long, and has a radius of gyration of 4.61" which gives a value $\frac{L}{r}$ of 7.37 and consequently P is 9,800 lb. The stress 88,000 lb. will therefore require $\frac{88,000}{9,800} = 9.00\ \square''$; and since the given area is 12.06 □" the efficiency is $\frac{12.06}{9.0} = 134\%$.

The connections to the chords at both top and bottom are by 24 $\frac{7}{8}''$ field rivets. The required number is $\frac{88,000}{3,680} = 24$ and therefore the efficiency is $\frac{24}{24} = 100\%$.

Efficiencies of Vertical Posts.

Mark.	Section.	Length	r	$\frac{L}{r}$	Allowable Unit St.	Total Stress	Required area	Given area	Efficiencies.
U_1L_1	2 [s 12×20.5	18.0	4.61	3.9	12,600	22,000	1.75	12.06	690%
U_2L_2	"	20.5	"	4.45	12,300	81,000	6.6	"	183.
U_3L_3	"	23.0	"	5.0	11,800	94,000	7.97	"	151.
U_4L_4	"	27.0	"	5.86	11,200	97,000	8.21	"	147.
U_6L_6	"	42.5	"	9.23	8,300	84,000	10.1	"	119.
U_7L_{7a}	2 [s 15×33	51.0	5.62	9.08	8,300	159,000	18.9	19.8	105.
U_7L_{7b}	2 [s 15×40	51.0	5.43	9.4	8,100	194,000	24.0	23.5	98.
U_8L_8	2 [s 12×25	46.0	4.43	10.4	7,200	95,000	13.2	14.7	111.
U_9L_9	"	43.5	"	9.82	8,400	79,000	9.4	14.7	156%
$U_{12}L_{12}$	"	42.5	"	9.6	7,800	100,000	12.8	14.7	115.
$U_{13}L_{13}$	"	43.0	"	9.7	7,800	105,000	13.4	"	110
$U_{14}L_{14}$	"	46.0	"	10.4	7,200	111,000	15.4	"	95.
$U_{15}L_{15}$	2 [s 15×40	51.0	5.43	9.4	8,100	208,000	25.6	23.5	92.
$U_{16}L_{16}$	"	51.0	"	9.4	8,100	214,000	26.4	"	89.
$U_{17}L_{17}$	2 [s 12×20.5	41.5	4.61	9.0	8,300	109,000	13.1	12.06	92.
$U_{18}L_{18}$	"	35.4	"	7.7	9,600	110,000	11.5	"	105.
$U_{19}L_{19}$	"	30.0	"	6.51	10,500	104,000	9.9	"	122
$U_{20}L_{20}$	"	26.0	"	5.64	11,300	102,000	9.07	"	133.
$U_{21}L_{21}$	"	23.5	"	5.1	11,800	100,000	8.47	"	142
$U_{22}L_{22}$	"	21.4	"	4.64	12,100	84,000	6.95	"	174
$U_{23}L_{23}$	"	20.0	"	4.34	12,400	22,000	1.8	"	670.

Art. 19. Compression Members - Box Girder

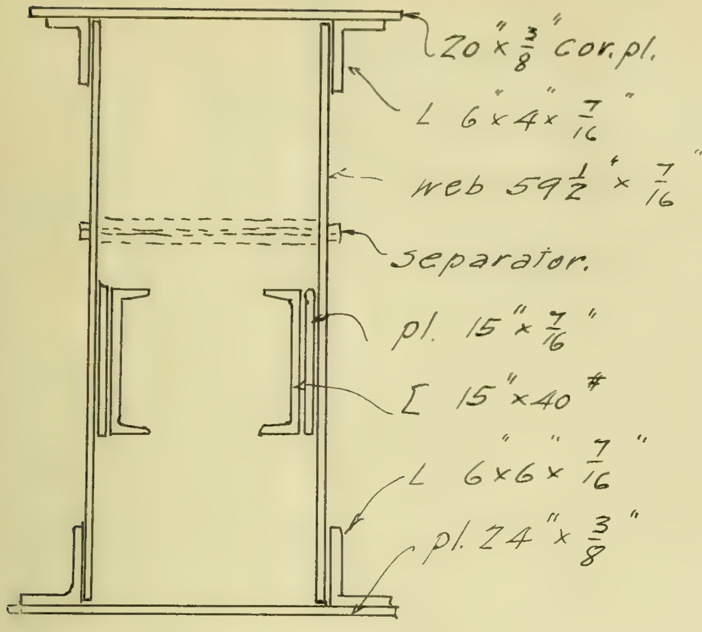


Fig. 18.

The composition of the box-girder under the fixed tower is shown in Fig. 18. The channels and plates on the inside were designed to carry the direct stress on the member, 364,000 lb.

The girder will be considered as carrying a uniformly distributed load equal to the reaction. For the maximum reaction, the trolley will be directly over the panel in question, or the total reaction will be the sum of trolley load, 75,000 lb. plus the dead load reaction 210,000 lb. or 285,000 lb. The moment of this is $\frac{Wl}{8}$

$$= \frac{285,000 \times 212.5}{8} = 7,570,000 \text{ lb. in.}$$

This moment is resisted by the moment of the flange-stress, F, which is F×h where "h" is the effective depth. In this girder, h will be assumed as 54", which makes F equal to

$\frac{7,570,000}{54} = 140,000$ lb. At a unit stress of 16,000, this stress would require an area of $\frac{140,000}{16,000} = 8.8\,\square''$ net in the flange. The given area is that of the cover-plate and the two angles. The upper flange is considered as that is the smaller. The gross area of the cover-plate is 7.5 □" and the net area = 6.75 □." The gross area of the angles is 8.36 □" which is reduced by rivet-holes to 7.49 □" net. The total net flange area is then 6.75 + 7.49 = 14.24 □". Since 8.8 □" are required, the efficiency of the flange is $\frac{14.24}{8.8} = 161\%$.

Art. 20. Floor Beams.

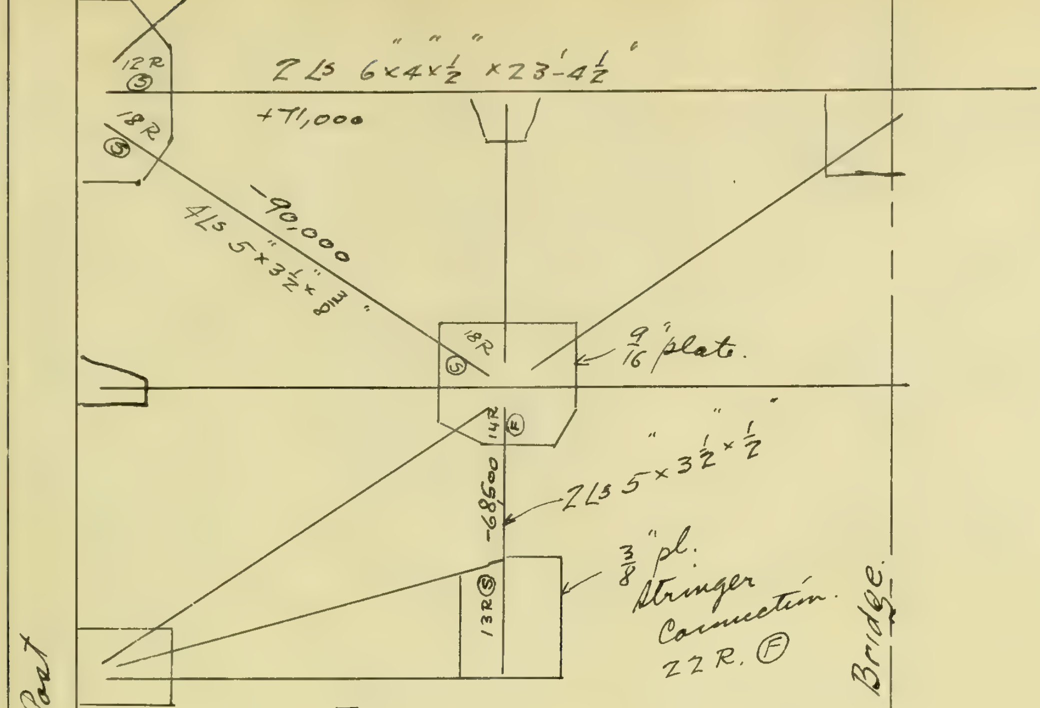


Fig. 19.

Note – All rivets 3/4" ⌀

(F) = Field rivets.

(S) = Shop "

The floor-beams are latticed girders, composed entirely of angles and gusset plates. Fig. 19 shows the parts in their respective positions, the stresses carried and the sizes.

The vertical member which carries the longitudinal stringers, is composed of 2 Ls 5"×3 1/2"×1/2" with a net area of 6.38 □". The stress which it carries, 68,500 lb., requires an area of $\frac{68,500}{16,000} = 4.28$ □" net. The efficiency then is $\frac{6.38}{4.28} = 149\%$. As shown by Fig. 19.,

the connection at the top is by 14 $\frac{3}{4}$" field rivets, while the required number is $\frac{68,500}{5,620} = 13$, 5,620 lb. being the value of a $\frac{3}{4}$" field rivet in $\frac{1}{2}$" bearing. In the lower connection there are 13 $\frac{3}{4}$" shop rivets; the required number is $\frac{68,500}{6,750} = 11$, 6,750 lb. being the value of a $\frac{3}{4}$" shop rivet in $\frac{1}{2}$" bearing. The efficiency of the upper connection is $\frac{14}{13} = 108\%$, and of the lower connection $\frac{13}{11} = 118\%$.

The diagonal member transferring the load from the gusset-plate to the vertical post, is composed of 4 L's $5 \times 3\frac{1}{2}" \times \frac{3}{8}"$ with a net area of 9.77□". The stress, 99,000 lb., requires a net area of $\frac{99,000}{16,000} = 6.2$□", so the efficiency of the member is $\frac{9.77}{6.2} = 157\%$.

The connections at both ends of the member contain 18 $\frac{3}{4}$" shop rivets which are in double shear. The required number will therefore be controlled by the bearing value in the $\frac{3}{8}$" leg of the angles, and is $\frac{99,000}{5,070} = 20$. The efficiency of the connection is then $\frac{18}{20} = 90\%$.

The horizontal strut between the vertical posts, carries the horizontal

component of the stress in the diagonal just investigated. This component is 71,000 lb. compression and the member must therefore be investigated as a column. It is composed of 2 L's $6'' \times 4'' \times \frac{1}{2}'' \times 23'\text{-}4\frac{1}{2}''$; the 4" legs are placed back to back with $\frac{5}{16}$" washers between; see Fig. 20.

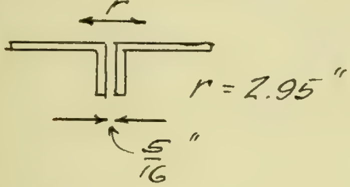


Fig. 20.

There are several braces for the 4" legs and therefore the dangerous radius of gyration is in the direction of the 6" legs. This "r" is found to be 2.95 inches. The value of $\frac{L}{r}$ is $\frac{23.35}{2.95} = 7.9$ which gives a unit allowable stress of 9,300 lb. The total stress, 71,000 lb., therefore requires an area of $\frac{71,000}{9,300} = 7.62$ □" and since the given available area is 9.50 □" the efficiency is $\frac{9.50}{7.62} = 124\%$.

The connection to the $\frac{9}{16}$" plate at the vertical post has 8 $\frac{3}{4}$" shop rivets in the $\frac{1}{2}$" legs of the angles and 4 $\frac{3}{4}$" shop rivets in the $\frac{5}{16}$" legs of two additional angles. The total value of these rivets is $8 \times 6,750 + 4 \times 4,220 = 70,800$ lb. and the efficiency, therefore, is $\frac{70,800}{71,000} = 100\%$.

Art. 21. Stringers.

The stringers are plate girders which support the track for the trolley on their upper flanges. The stringers are all of the same depth, 2'-8½" and are very nearly alike, the only difference being, that in the two longer panel lengths, 21'-5" and 22'-8¼", the flange angles are $5 \times 3\frac{1}{2} \times \frac{9}{16}$" while in the 20'-9½" panels the flange angles are $5 \times 3\frac{1}{2} \times \frac{1}{2}$". The 22'-8¼" stringer will be investigated.

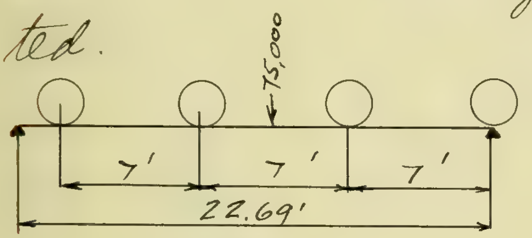


Fig. 21.

The maximum end shear will occur when the trolley is in the position shown in Fig. 21, and is $\frac{22.69 - 10.5}{22.69} \times 75{,}000 = 40{,}400$ lb. The dead load shear is that due to weight of the stringer and its bracing, 1,300 lb. or a total of 41,700 lb. The required web area is $\frac{41{,}700}{10{,}000} = 4.17$ □" and since the depth is 32," the thickness required is $\frac{4.17}{32.0} = 0.13$." Since a ⅜" web was used the efficiency in shear is $\frac{0.37}{0.13} = 284\%$.

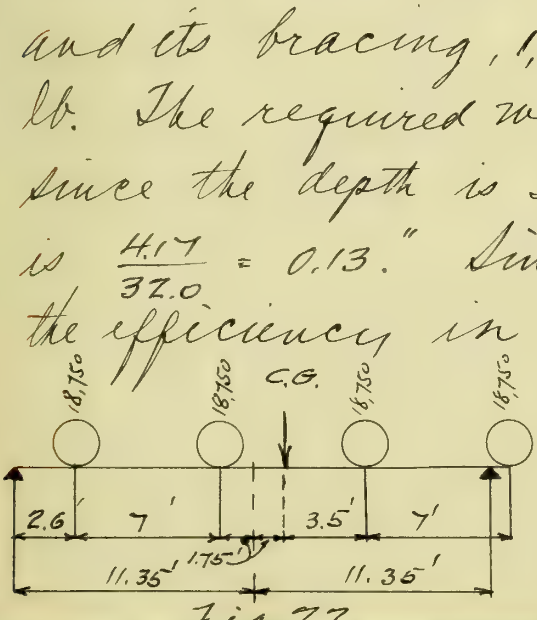


Fig. 22.

The bending moment of the loads is the sum of dead and live load bending moments. The position

of the trolley for maximum bending moment is shown in Fig. 22. This bending moment is $- + (1.75 + 8.75)\,18{,}750 - \left(\frac{9.6}{22.69} \times 75{,}000\right) 11.35 = 107{,}000'^{\#}$ The dead load bending moment is $\frac{Wl}{8} = \frac{2{,}400 \times 22.69}{8} = 6{,}800'^{\#}$ or a total of $107{,}000 + 6{,}800 = 113{,}800'^{\#} = 1{,}366{,}600.''^{\#}$ The depth of the girder is $32\frac{1}{2}''$. The effective depth may be assumed as 28", which gives a flange stress of $\frac{1{,}366{,}600}{28} = 48{,}800.^{\#}$ At a unit stress of 10,000 lb. per sq. in., the area required is $\frac{48{,}800}{10{,}000} = 4.88$ and since the net area of the flange is $6.97\square''$, the efficiency of the flange is $\frac{6.97}{4.88} = 142\%$.

Conclusion.

The conclusion that may be drawn from the investigation of the members of the gantry-crane, is that some are below the requirements as to area and some are deficient in connections. These deficiencies, however, are not so dangerous as might appear, when it is remembered that the assumed live load of 75,000 lb. per truss is from 15,000 to 20,000 lb. heavier than the actual live load will ever be. The factor of safety also, will take care of the few deficiencies, so that it may safely be said, that the bridge is all right. Another cause for such a belief, is the fact that the bridge has been up and in successful operation for several years.